Pavements - Design
Pavements -- Failures

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PAVEMENT DEFLECTIONS and FATIGUE FAILURES

By

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In 1948, a paper was presented at the 28th Annual Meeting of the Highway Research Board wherein an attempt was made at identifying and classifying the numerous factors and properties of materials that singly or in combination affect the performance of highway pavements.(1) A chart was used to analyze the relationship between the major and minor factors. While the original paper attempted to include all of the factors in the discussion under Part 1, "Analysis of the Pavement Design Problem," the solutions, test methods and design chart proposed at that time were aimed at providing answers to only two of the three primary basic problems shown on Fig. 1. In other words, the design procedures then proposed and which have since been followed in California and subsequently adopted by several other states and foreign countries are confined to anticipating the ultimate density, the amount of moisture which could ultimately be taken up by the soil and the resistance value of the soil and base material when the worst conditions will have been reached.

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This procedure, however, does not provide safeguards against failure in the form of cracking or breaking up due to fatigue resulting from continual flexing or bending of the pavement structure under passing wheel loads, Fig. 4. This third factor has long been recognized and appears as item three in the second column of Figure 1, which is reproduced here.

ANALYSIS OF THE PROBLEM

In order to simplify and further illustrate the relationships between the several factors and the structural adequacy of pavements, Figure 2 presents in tabular form the same three basic problems together with the properties of pavements, bases and soils which must be recognized and reconciled in order to provide an answer to each problem and thus produce an all-round satisfactory pavement. In Figure 2 the three problems are listed at the head of the columns numbered 1, 2, and 3, while in the left-hand column the pertinent properties of the basement soils, bases and pavements are listed in two separate groups. This arrangement is intended to indicate that the engineer must generally accept the basement soil with its inherent properties as it will exist in the roadbed. He must evaluate these properties by suitable tests in order to assign numerical values to the important variables. It will be noted that the basement soil whether in situ or imported is considered to have four important properties which must be determined by separate tests and evaluated independently. These are:

A. Internal Friction
(R-value, measured by the stabilometer)

B. Cohesion
(Tensile resistance, measured by the cohesiometer)

C. Swelling Pressure
(Expansive force exerted during the absorption of water, measured by expansion pressure device)

D. Resilience
(Compression and rebound under passing loads, measured by resiliometer)

As stated before, while some selection is often possible the engineer must, in general, accept the basement soil and deal with the properties as they will exist beneath the pavement after the passage of time, perhaps of several years. However, the engineer has greater freedom in choosing or providing the properties and dimensions of the pavement and the base materials. (The exercise of this choice is often called "Engineering Judgment.") The most important of these properties is set forth in Figure 2 as

a. Flexural Strength
(Variously referred to as beam strength, slab strength and may be indicated by modulus of rupture values, tensile strength test or by the cohesiometer.) This quality contributes to the "stiffness" of the pavement.

The second important contribution by the pavement and associated base layers is

b. Direct weight resting on the subgrade (Easily determined by computation knowing the proposed thickness and the unit weight of each layer.) The "stiffness" factor of the pavement and base combination will also vary with thickness even of granular masses of low cohesive strength.

and the third is

c. Flexibility (Fatigue resistance) (Ability to withstand repeated bending or flexing. The over-all flexibility is influenced both by the thickness of the pavement section and by the elasticity. A new device is now under development in the laboratory to evaluate this property in terms of fatigue resistance.

In order to indicate how these various properties may affect the over-all performance, on Figure 2 the index letters are transferred and arranged to indicate "answers" in the columns representing the three basic problems. Thus, it will be seen that problem number one, "how to determine the ultimate moisture and density equilibrium," requires a knowledge of the potential expansion pressure of the soil as it will exist after construction operations and this tendency to expand can only be counteracted by placing a sufficient weight of cover (pavement plus base shown as "b" in Fig. 2) over the soil to balance or oppose the expansion pressure.* The second problem which is to prevent failure from plastic distortion of the underlying material requires that the internal friction be measured and perhaps the cohesive resistance also, although this latter can usually be neglected for design purposes. The expansion properties are significant here only because they may influence the amount of water taken up and thus indirectly affect the internal friction. There are two solutions to this problem, one, "a" to use a pavement of high flexural strength; the other, "b" to place a sufficient weight of base and surface, or, as there are no weightless pavements, some combination of the two is employed. As stated before, procedures for dealing with problems one and two have already been set up and are being followed in a number of highway laboratories today. However, so far as is known there have been no organized attempts to deal effectively with problem number three which seems to be increasingly serious in recent years due to the great increase in the weight and numbers of heavy wheel loads on highway pavements. Therefore, this paper will deal primarily with pavement deflections under repeated load applications and the resultant fatigue failures. Referring to the concept illustrated by Figure 2 it will be noted that problem three adds increasing complexities. For problem number three it now seems that the internal friction or resistance value of the soil may not be directly significant and cohesive resistance probably plays only a minor part. The main consideration is the actual

*Expansive tendencies can, of course, be reduced or eliminated by adding more water during or immediately following construction.

resilience* of the soil in the condition of moisture and density that will be characteristic of the materials after the pavement has been in place for some time.

The lower half of column three indicates that there are three possible answers or solutions to the third problem. If a pavement of sufficiently high slab strength is employed, "a" then it will not be deflected beyond safe limits by the passing loads. Also, if a sufficient weight or thickness of stable granular material "b" is used in the base course there will be no undue flexing of the pavement surface. Either or both types may have the required "stiffness."** And finally, at direct variance with the

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*"Resilience" is preferred to such terms as elasticity as we are here concerned with movements much greater than would be developed in many elastic solids such as glass, concrete, steel, etc. A new device termed the Resiliometer has been developed to measure this property of soils on laboratory specimens.

**The term "stiffness" has been borrowed from a report by L. W. Nijboer and C. van der Poel (2). Nijboer computes stiffness from the formula

$$S = \frac{F_p}{X_p} \quad (12) \text{ for } F_p \text{ equal to } 10^4 \text{ N (1 ton) and}$$

2×10^4 N (2 tons) respectively.

F_p = Force acting on pavement

X_p = Deflection of the pavement

Therefore, the term "stiffness" bears a simple mathematical relationship to the deflection of the pavement and as used by Nijboer "stiffness" implies the resistance of all components including the pavement, bases, subbases and the underlying soil. For design purposes it seems preferable to us to associate the concept of stiffness with the pavement and base structures alone in which case there will not be a consistent relationship between "stiffness" and "deflection" as the character of the supporting soil will then represent a variable - "resilience."

limited solutions for problems one and two, a thin flexible pavement "c" may serve quite well over resilient soils where a heavier, stronger but more rigid or brittle type will crack and perhaps show other signs of failure. However, the materials engineer who must recommend an adequate over-all design must make sure that whatever combination of pavement and base is proposed will adequately and simultaneously satisfy all three of the basic problems enumerated. In many cases thin pavements will not satisfy problem one or two. In view of the fact that methods dealing with problems number one and two were set forth in the 1948 paper(1) above mentioned and have subsequently been improved, the following will be confined to a discussion of the factors that must be taken into account for a solution to problem number three -- how to prevent fatigue failures in the pavement due to flexing caused by alternate depression and rebound under moving wheel loads.

DEFLECTIONS

For the purposes of this discussion, the term deflection will be used in a somewhat limited and special sense to indicate those movements of the pavement under traffic in the form of downward bending beneath the vehicle wheel followed by rebound after the load has passed on. For this purpose the term deflection applies to transient movements and is considered to be only one of several types of deformation which a pavement may undergo. Figure 3 presents an outline of the terms together with contributory causes which are subdivided into a primary and secondary group. For the purpose of this discussion the following definition applies:

> Deflection - A transient downward movement of the pavement when subjected to vehicle wheel loads. A deflected pavement rebounds shortly after the load is removed.

Pavement deflections have been a matter of interest to the writer and to this department for many years. While serving as a maintenance superintendent in 1924 prior to the use of bituminous surfacings on California's rural highways, it was observed that there were marked differences in the difficulty of maintaining untreated gravel roads and in many cases the behavior apparently bore some relationship to the character of the foundation soil. In other words, there were a number of examples on level grades where the gravel surfacing would remain in relatively good condition in cut sections where the roadbed consisted of solid rock as compared to shallow fill sections of fine grained soils. The troubles noted were chiefly in the form of raveling and "potholing" of the surface but as the surfacing material was the same throughout it seemed probable that there was a greater magnitude of flexing and bending under heavy wheel loads where the underlying soils were more resilient. Thirty years ago much less attention was given to the selection of materials in the roadbed, and such material as leaf mold and vegetable matter was not always rigorously excluded as, we hope, is the case today.

In 1938, the Laboratory of the California Division of Highways secured a General Electric Travel Gauge unit primarily for the purpose of measuring deflections of pavements under rapidly moving wheel loads. This unit was used in scattered investigations throughout the years both on state highways and for test pavements constructed by the State and the Corps of Engineers. Among other things, it was found nearly fifteen years ago that asphaltic pavement deflections varied greatly with temperature. However, it was not until 1951 that an organized study was undertaken to determine the actual deflections that traffic was inflicting on California highway pavements. The data furnished herewith are intended to be a progress report of a study which is by no means completed. The data presented herewith represent selected examples from 43 projects involving the installation of nearly 400 gauge units and over 2500 individual gauge records.

In 1951, a newly constructed section of asphaltic concrete pavement (less than two years old) was showing marked evidence of distress in the form of extensive cracking of the type usually described as "chicken wire" or "alligator cracking", Figure 4. This section of road is a four-lane divided highway on US 99 north of Los Angeles which is the principal truck route between Los Angeles and points north. The cracking was first observed and became most pronounced in the outer lane which carries over 80 percent of all traffic and virtually all of the heavily loaded trucks. However, on a short stretch of this project all lanes of the pavement remained in good condition with no evidence of cracking. Deflection gauges were installed in both cracked and uncracked areas and the deflection of the surface was measured with reference to rods driven through the base and anchored in the underlying soil at various depths. Figure 5 illustrates a typical installation of these gauge units. Figure 6 is a plot of the deflections which were measured on this project against reference rods 3-ft. long using axle loads ranging from 11,000 to 29,000 lb. as indicated by the abscissa scale. It will be observed that there is a marked difference in the magnitude of the deflections measured where the pavement is badly cracked compared to the area where there is no evidence of cracking. Most of the deflection measurements that have been made to date indicate that in general the measured deflection bears a linear relationship to the load applied although this relationship does not everywhere hold true. As will be shown later certain types of soil especially where deflections are high develop a distinctive load deflection pattern that is not in accord with the more or less linear relationship indicated in Figure 6.

In any event, it will be apparent that the badly cracked portions of the pavement have been continuously subjected to much higher deflections than has the section where no cracking is in evidence. It might be argued, of course, that the deflections are higher because the pavement has cracked and thus lost continuity and slab strength. Undoubtedly, the deflections are greater when the slab continuity has been destroyed; however, the deflection in the passing or inner lane at the same location is also relatively high compared to most uncracked pavements and the absence of cracking in the inner lane can only be attributed to the fact that it carries relatively few heavy loads. The structural section used on this project is shown in Figure 7. It will be noted that it is as heavy and substantial as that used on the New Jersey Turnpike - for comparison. The asphalt concrete surface has a stabilometer value of 45. The crushed granite base has an R-value of 77 and a CBR of 161, the subbase has an R-value of 74 and a CBR of 125. However, it now appears that the basement soils on this project are definitely resilient and there is also evidence that the asphalt has become hardened and, therefore, the pavement is more brittle than desirable, The original asphalt had a penetration between 120 and 150, however, recoveries of the asphalt made in 1951 by the Abson method show an average penetration of 36 after one year on the road. The records indicate that plant temperatures were quite moderate and unusually well controlled ranging between 275 and 285 F. While subjected to heavy traffic and badly cracked as noted above and shown by Figure 4 - this pavement has remained smooth and undistorted indicating that the "failures" were entirely due to flexing and bending.

The second project that was investigated is on the Bayshore highway, US 101, one of the heaviest traveled routes in the state, being the main traffic artery from San Francisco south. A portion of this highway was reconstructed on new alignment in 1947 traversing an area of mud flats which required imported embankment materials up to 25 feet in depth surfaced with five inches of asphaltic concrete resting on an 8-inch depth of crushed stone base over 24 inches of sand subbase. Failures in the asphaltic surfacing in many areas on this section, Figure 8, became evident within one or two years after construction. As an investigation conducted by the laboratory could discover no deficiencies in the quality of the asphaltic pavement or of the base material, it was decided to measure the magnitude of deflections. Figures 9, 10, 11, and 12 illustrate the deflection measurements made on this project using reference rods of different lengths. Figures 9 and 10 represent the deflections at two locations where the pavement is in good condition.* Figures 11 and 12 represent readings taken at points where cracking and distress of the asphalt pavement were very evident. Figure 13 gives a profile view of the measured deflections illustrating the length of pavement involved in these deflection "zones." Attention is drawn to the evidence that truck loads can affect the pavement foundation to a depth of 18 feet or more. It will be obvious, of course, that the magnitude of vertical deflection is not of itself completely significant as the tendency to break or rupture the pavement will depend primarily upon the sharpness of the arc or curvature of the pavement surface. Engineers in Sweden have concluded that when the pavement is bent in an "arc" having a radius less than 100 feet, failures would result. It may be that asphalt pavements in Sweden are more flexible; in any event we have been unable to arrive at a similar value for California conditions although the general idea seems sound. While the vertical deflection measured with reference to a rod 18-ft. in depth is of the greater magnitude, it may well be that the most severe

*One of these was in an area where vertical sand drains had been placed in the embankment. This poses an interesting question about the function and performance of this currently controversial type of installation. There is no evidence that the fills have settled more rapidly with the sand drains than adjacent areas without this provision. Borings alongside the sand columns brought no evidence of dewatering of the soil. Nevertheless the pavement is in better condition and as stated above the deflections are markedly lower. Perhaps the 3-ft. layer of pervious sand placed as a blanket over the vertical drains is the answer?

stresses in the pavement are associated with compression and rebound in the upper layers of the embankment which may produce sharper bending and consequent greater stress in the pavement slab. In other words, the shape of the depressed area will undoubtedly vary with variations in both basement soil and pavement structure. It must also be recognized that a moving wheel load causes a sharp reversal of stress from tension to compression in every portion of the pavement in the wheel path. There is evidence to indicate that in many cases the sharpest bend and consequently the greatest stress is outside the wheel contact area.

After the initial studies outlined above, it was tentatively assumed that the compressibility and rebound in the top eight or nine feet of the roadbed soil would be of greater significance and more readily correlated with performance than the total over-all deflection. Therefore, on the more recent work, these depths have been used for purposes of comparison although it is readily admitted that the question of how best to employ a simple measurement of deflection as an index of destructive bending movements has yet to be settled in our minds. However, in order to "start somewhere" we have compared deflections referred to an eight or nine foot reference rod under 15,000 lb. single axle loads. This means that what we are actually reporting is the compression and rebound of the soil in the upper eight or nine feet of the roadbed.

An asphaltic concrete pavement on US Route 40 between Sacramento and San Francisco has given an excellent performance since 1937. Selecting a section that was definitely in good condition, deflection measurements were made as shown on Figure 14. Here it will be noted that the compression and rebound in the top nine feet of the roadbed under a slow moving 15,000 lb. axle load is 0.012-in. In this instance we have also shown for comparison the deflection caused by a static or standing load. While the difference here is greater than most it is true that deflections under static loads are always greater than under moving loads. By way of comparison, Figure 15 refers to a portland cement concrete pavement five miles north of Eureka. This pavement is some 28 years old and in fair condition considering its age, type of foundation and amount of traffic. Here the deflection related to the upper nine feet is moderate, being 0.016-in. Next a few asphaltic surfaces on the Redwood Highway were studied where the roads are subjected to heavy log hauling. One section across swampy ground (Beatrice Flats) has undergone periodic settlements throughout the years and as a result the roadbed has been built up by additional layers of gravel and bituminous surfacing until at the present time the total thickness of gravel is over 30 inches. The asphalt surface is now in excellent condition and has remained so for some time. Again one may note from Figure 16 that the deflection with reference to a

9-ft. rod is 0.017-in. An even more striking example is the present road on the Redwood Highway south of the town of Scotia which was reconstructed in 1946 using a 3-inch plant-mix surface over an 8-inch cement treated base supported by a substantial subbase of pit run gravel. Here the deflections under a 15,000 lb. load for a 9-ft. depth are only 0.009-in., Figure 17. The excellent appearance of this surface and freedom from maintenance cost over a period of eight years testify to the fact that this pavement is adequate for the heavy traffic which it must sustain. In marked contrast are the very high deflections measured on a section of old secondary road 21 miles southeast of Eureka where the old Warrenite pavement shows evidence of extensive cracking. This section has been resurfaced several times and the maintenance costs have been high. Figure 18 illustrates deflections as high as 0.140-in. One might comment at this point that damage to a pavement is not necessarily in direct proportion to the magnitude of the deflection. Once the pavement is cracked into small blocks it then acquires the ability to bend and the "articulated" structure can accommodate considerable movement without necessarily progressing rapidly to complete failure. Such cracked pavements do not "look pretty" and are a worry to the engineer - however, in many cases they will carry traffic for a long time proving that true flexibility is a desirable characteristic if it could be obtained with a tight uncracked pavement.

Figure 19 shows a comparison between the deflections measured in a pavement supported in one area by a cement treated base and in another by a gravel base. The gauges were set in the outer wheel track of a traveled way which was supported by 6 inches of cement treated base. Gauges were also placed in the adjacent shoulder section which was surfaced with plant-mix resting upon 19 inches of pit run sand and gravel containing appreciable amounts of clay and silt. Here it will be noted that the cement treated base apparently has a very marked effect in reducing the pavement deflections. Additional confirmation of the effect of slab strength is shown by Figure 20 which illustrates the very low deflections of a section where 4½ inches of plant-mixed surfacing is supported by eight inches of cement treated base. This section is on the main highway known as the Castaic Bypass about 43 miles north of Los Angeles. The appearance of this pavement is excellent with only an occasional transverse shrinkage crack. The use of cement treated bases appears to be an effective means of reducing the deflections in many cases.

However, there is also evidence that deflections may be reduced to equally acceptable limits by means of heavy gravel or crushed stone bases as is illustrated by Figure 16 and Figure 21. Figure 21 shows the same comparison as Figure 19, but in this case the difference in deflection between the pavement resting upon a cement treated base and the adjacent shoulder supported only by sand and gravel is very small. The excellent condition of both shoulder and traveled way are in accord with the very low deflections measured.

A number of deflection measurements have also been made on concrete pavements, most of which are in relatively good condition however. Some of these curves showing deflection versus load are arranged on Figure 22. It is a difficult matter to make comparisons between the deflections of a concrete pavement and those of a bituminous type. The deflections will vary throughout the length of the average concrete slab, which at night or in the early morning is usually curled up at each end thus losing contact with the subgrade. The deflections, therefore, are generally greater at the ends of the slabs than at the midpoint, and this curling or warping is affected by both temperature and moisture. Therefore, in order to measure deflections which reflect the bending of the slab due to compression and rebound of the subgrade, it is necessary to place the gauges in the slab midway between the joints. In any event it will be noted that the deflections are all comparatively low for concrete pavements in good to excellent condition. It must be pointed out, however, that the ends of most concrete pavement slabs are being continually flexed under passing vehicles not because of subgrade compression but because the ends are often unsupported for a distance of five to seven feet from the joint. Therefore, failure and breakup of concrete pavements may, in many cases, be unrelated to subgrade compression and rebound. Obviously, the problem of measuring and evaluating pavement deflections refuses to remain simple.

As an aid in visualizing the shape of the depression "basin" in the pavement a three dimensional model was carefully constructed to an exaggerated scale. Figure 23 is a photograph of this model representing a typical deflection pattern of a bituminous pavement on a gravel base under a 9,000-lb. dual tire wheel load.

PAVEMENT CONDITION VERSUS DEFLECTIONS

A few examples were found where pavements undergoing fairly high deflections appeared to be in good condition as indicated by some of the solid lines on Figure 25, but it will, of course, be obvious that asphaltic pavements inevitably vary somewhat in their ability to withstand repeated flexing without evidence of cracking, Figure 32. There are differences in the aggregate gradations, in the grade and amount of asphalt. There are differences in ages of the pavements, in the thickness and differences in the climatic temperature range. As mentioned above, most of the deflections shown thus far have indicated an approximately linear relationship with load, that is, the magnitude of the deflection is in direct proportion to the magnitude of the load. However, in one area of the state;

namely, on the coastal highway near San Luis Obispo, marked cracking of the pavement has been observed on two separate contracts separated by a few miles, but utilizing similar materials in the imported subbase layers. A plot of all deflections measured on these sections shows a characteristic curved pattern, that is, concave downward indicating that the deflections are disproportionately high for the lighter loads. Nevertheless, it is true here as elsewhere that the measured deflections under 15,000-lb. axle loads in areas where the pavement is in good condition are generally less than 0.020-in. Deflections made in cracked and failed areas on these sections generally exceed 0.025-in. Figure 24 shows deflection measurements on this section.

The data shown herewith represent only selected examples of a large number of readings that have been made over California pavements. Figure 25 is a summary chart showing a comparison between the deflections characteristic of cracked pavements compared to those found where the surfacing is in good or excellent condition.

Deflection measurements have been made by the Corps of Engineers on airport pavements and by the Bureau of Public Roads and the Highway Research Board on the experimental test tracks of One-MD in Maryland[3] and on the WASHO track in southern Idaho.[4] There has not been an opportunity or time to compare all of the available deflection data in order to establish general laws or rules, and knowing something of the variations which may exist in asphaltic paving mixtures it would be unwise to make too positive statements at this time concerning the amount of deflection which an individual pavement in a given area and climatic environment may safely withstand. However, considering that the study is incomplete and that further evidence may cast a different light on these conclusions it now seems clear that the type of dense graded asphaltic pavements frequently constructed in California (approximately three inches thick) will not long endure repeated flexing that exceeds 0.020 of an inch and heavier pavements appear to be limited to even lower values.

It must be pointed out that this study does not undertake to say exactly how much deflection is being produced by the current truck traffic passing over the road. As the failures are a fatigue phenomena the cracking is the result of both the magnitude of bending or flexing and the number of repetitions. Most highway traffic represents a distribution or range of loads and while a moderate number may reach or even exceed the 18,000-lb. axle load limit it is evident that the "average" wheel load must be somewhat less. Therefore, we have a complex problem in evaluating traffic where a few heavy loads cause high deflections and the many lighter loads cause lower but more numerous bending repetitions. As stated before, we have assumed that an over-all summation would be equivalent to an equal number of repetitions of around 15,000-lb. axle load. On this assumption the cracking and fatigue failures of most pavements are attributable to a large number of deflections having an effective equivalent greater than 0.020 or 0.025 inches. Thinner or more flexible pavements would obviously raise this limit of tolerance or in the absence of heavy loads the pavement would not be subjected to the magnitude or number of bending stresses.

VIBRATIONAL METHODS VERSUS DEFLECTIONS UNDER SLOW MOVING LOADS

Thus far we have discussed pavement deflections produced under slow moving truck loads and the apparent relationship between these deflections and the condition of the pavement surface. The study has many ramifications and interesting "by-products" that have not been touched upon. For example, we were able to undertake some comparisons between the deflections caused by slow-moving loads on pneumatic tires and those developed by vibrational means. Through the courtesy of the Shell Oil Company, the vibration tester developed in Holland(2) was made available and measurements of strain, deflections and velocity of wave propagation were taken during July 1954 at a number of locations on California highways where electronic gauges had been previously installed and earlier readings secured. Attempts to establish a correlation between the deflections under wheel loads and those produced by the vibration machine were not too successful, probably for several reasons. One, the lapse of time between the GE gauge readings and the measurements with the vibration machine, and two, the fact that the vibrator operates through a heavy superimposed dead load which probably tends to suppress some of the amplitude of movement. Some comparison was made between deflections indicated by the Shell Company vibrator and those obtained by the Benkelman beam [4] on the same day. While it is apparent that no close correlation exists, nevertheless there is a general relationship even though the range of values obtained by the Benkelman beam under a 9,000-lb. wheel load is greater than the dynamic deflections registered by the vibration machine developing a force equal to two metric tons. Figure 26 shows a Benkelman beam being used to measure deflections caused by a heavy wheel load. In general, it appears that there is no major difference in the evaluation of pavement stiffness which would be arrived at by either of the two methods, and the low cost, simplicity and speed of operation with the Benkelman beam device makes it a very attractive instrument for the initial study of pavement deflections. The Benkelman beam does, however, have the limitation that it is impossible to identify the layer of material beneath the pavement that is responsible for such deflection as may occur. For this purpose we have found no substitute for the electric gauge units which make it possible to install a series of reference rods of varying lengths and thus identify the layer or horizon beneath the pavement that is chiefly responsible for the compression and rebound action. The GE gauges also avoid a possible error that might result with the Benkelman Beam where the length of pavement depressed is greater than 8-ft, Figure 13, for example.

As stated before, work in this laboratory has not progressed far enough to permit setting up a laboratory procedure giving information which will enable the designer to anticipate conditions of resiliency in the basement soil. Work on these lines is under way and we now believe we are justified in feeling optimistic about the outcome.

NEW TESTS AND DESIGN PROCEDURES

It appears that there are three major subdivisions of the laboratory work and the analytical steps necessary to develop a solution for this problem. First, is the measurement of deflections which are characteristic of existing pavements. This investigation requires many measurements over as wide a variety of pavement types and conditions as possible. From this study it should be possible to establish the magnitude of deflection which is characteristic of the failed sections compared to the amount shown by pavements in good condition. This work is under way in California and some of the preliminary results have been discussed and illustrated in the first part of this report.

In order to utilize the findings in the daily work of highway and airport engineers and to develop more realistic designs, it will be necessary to have laboratory means for evaluating the potential resilience of soils and proposed foundation materials and <u>to be of any use for the average highway program such tests must be performed on samples taken in advance of design and of course even further in advance of actual construction.</u>

MEASURING RESILIENCE OF SOILS

The first model of a Resiliometer was developed and constructed in this laboratory in 1946. Preliminary trials indicated that it was possible to measure differences in the compression and rebound characteristics of soils, but work on the device was sidetracked for a time due to pressure of other projects. An improved model was constructed in 1953 and work was well under way until interrupted by a fire in the laboratory in March 1954. Since that time, Resiliometer model number three has been designed and constructed, Figures 27 and 28, and for the first time results appear to be consistent, and it now seems that it will be possible to measure and evaluate potential resilience using the small standard Stabilometer specimens 4-in. in diameter and 2½-in. in height for the purpose. Attempts to develop a satisfactory laboratory device and technique are only well started, and it would be definitely premature to make positive statements or attempt final conclusions at this time. For example, it is not yet clear what pressures should be used in the Resiliometer cycle in order to subject specimens to forces of the same order of magnitude as the pressures transmitted to the basement soils through the overlying layers of base and pavement. Obviously, of course, in the actual roadbed the pressures will vary with depth but for practical routine testing purposes it would be much more convenient to deal with a single figure for resilience using a single standard pressure in the test apparatus.

Tentatively therefore, we think that a pressure of 20 psi may be about right. Charts, Figures 29 and 30, show readings obtained with the Resiliometer. Resiliometer "readings" are in terms of the volume of displacement or compression and have not yet been correlated with linear units of pavement depression.

Figure 29 is an expansive resilient soil from southern Idaho and this graph illustrates rather vividly the fact that resilient properties are not manifest until the voids in the soil are filled with water, after which the susceptibility to compression and rebound increases rapidly with further increase in the moisture content. It is easily demonstrated, of course, that an expansive soil will take up moisture well beyond the point usually referred to as "maximum density" and "optimum moisture" content. Figure 30 illustrates Resiliometer measurements on samples of well graded gravel. Here the addition of moisture tends to diminish even the small amount of resiliency that exists. While several details of technique and laboratory procedure are still to be settled, these results seem to warrant the belief that a successful test procedure can be evolved. It will be observed that the magnitude of deflection and rebound increases with increasing moisture content after a certain value has been exceeded and also increases with increasing unit pressure. These preliminary results strongly suggest that the flexing of pavements under passing wheel loads may undergo a sharp increase in magnitude as soon as the subgrade moisture reaches the saturation point. This is especially true of the agricultural soil types containing appreciable amounts of fine materials or clay and probably entrapped air. As granular materials such as clean sands, gravels or crusher run bases characteristically show very low values in the Resiliometer it is beginning to appear that there may be a closer correlation and parallelism between results in the Stabilometer and measurements in the Resiliometer than was first expected. Before Resiliometer test results were available the idea was entertained that when the soil pores were filled with water, the combination would be virtually incompressible and consequently the resilience ought to be very low. Actual tests, however, have shown that if the soils do display any appreciable resilience the range of movement increases with increasing moisture content beyond the point where the voids are first filled with water. Obviously, any granular mass cannot contain more water than the void space will accommodate but while this void space is comparatively stable and fixed for a clean sand or gravel, such is not the case in fine grained soils of the expansive type. Here the capacity for moisture will increase markedly as the soil expands. On such soils the internal resistance (R-value) will diminish but it appears that the springiness or resilience will increase. While the foregoing trends seem fairly evident, a great deal of work is yet to be carried out before the Resiliometer becomes a proven device for the routine testing and evaluation of the resilient properties of soils.

FATIGUE RESISTANCE OF ASPHALTIC PAVEMENT

A second evaluation which must be made in the process of rational design concerns the ability of various types of pavement to withstand continued bending and flexing under the repeated action of traffic. How flexible is a "flexible pavement"? This characteristic will be more difficult to evaluate individually by test on pavement samples performed in advance of actual construction. In the first place it will be very difficult, if not next to impossible, to manufacture laboratory specimens that will have all of the properties of aged asphalt pavements that have been under traffic for some years. At the present time it seems questionable whether satisfactory and truly representative specimens can be formed in the laboratory for the purpose of measuring pavement flexibility. Fortunately, it does not appear that such a procedure will be absolutely necessary as it should be possible to establish characteristic limiting values typical of pavements which have been in use for several years. It is well-known that asphalts tend to harden with the passage of time; therefore, asphalt pavements are undergoing constant change in their properties because of oxidation, loss of volatiles, polymerization and increasing density under the action of traffic. It seems that any evaluation of the ability of an asphalt pavement to withstand fatigue failures must be based on observations of actual performance on the road. Characteristic safe values can be set up for design purposes as is common practice for all structural materials. However, it should be possible to confirm evidences of performance by the use of beams or test specimens sawed from actual pavements.

Reports of work at the University of Illinois on the fatigue of portland cement concrete have indicated that a sufficient number of repetitions of a load that equals only 50 percent of the modulus of rupture value will ultimately cause failure.(5) The modulus of rupture value is not easily or accurately determinable on a ductile, yielding material such as an asphalt pavement specimen. However, studies have been made on the fatigue resistance of small asphaltic pavement beams in the University of Washington under the direction of Professor R. G. Hennes.[6] Also recent reports from Sweden indicate an interesting relationship between tensile strength and temperature.[7] Studies are now under way in this laboratory with a newly constructed device for measuring the fatigue resistance of typical asphalt pavements, Figure 31. Only a few results are available at the time of writing this paper and the device for measuring fatigue or flexibility of bituminous pavements will undoubtedly undergo some changes and improvements, chief of which will be means for maintaining accurate temperature control. However, the initial trial results are interesting and Figure 32 illustrates the results obtained on small beams of asphaltic pavement cut from slabs taken from a road surface. These preliminary results are unexpectedly uniform and show a definite relationship between the magnitude of deflection and the number of repetitions required to produce failure. It will take some time to establish the relationship between deflections of these small beams and the deflection of a pavement slab under heavy wheel loads, but there is little room for doubt that such a relationship does exist. Curve A on Figure 32 represents an uncracked pavement. Curve B represents beams from a cracked pavement. It is evident that the quality of the pavement is one of the variables.

In connection with the work of the WASHO test road, Mr. William Carey of the Highway Research Board had been experimenting with a device to apply sudden rapid loads to beams of an asphaltic pavement. He suggested that we might do some work along the same lines and also suggested the use of a Soniscope. Soniscope tests have been performed in our laboratory with some interesting results tending to confirm the work reported from Sweden(7); however, we believed that it would be necessary to subject pavement specimens to repeated loading in order to simulate actual road conditions. Hence, the device for measuring fatigue susceptibility in asphalt pavements was constructed with a cam arrangement operating at speeds to simulate the sequence of wheels on a multiple axle truck. In the preliminary trials, small beams 2- by 2- by 10-in. cut from asphaltic pavements have failed by cracking after being deflected 0.008-in. repeated 12,000 times at a temperature of 75 to 78 F. While this magnitude of deflection on a short beam can not as yet be compared directly to the deflections measured on the roadway, Figure 13, there can be little doubt that such pavements would fail after fewer repetitions when temperatures are lowered to the range typical of winter conditions throughout most of the United States.

RATIONALIZATION OF THE DATA

In the light of the foregoing, it may be visualized that in addition to the design chart suggested in 1948[1], a second design process will need to be established in order to provide a sufficient depth or strength of pavement which will reduce the deflection to a value which the pavement can successfully tolerate throughout its entire economic life, or find means for constructing a flexible pavement that will not be damaged by the magnitude of bending stresses involved. It appears that we can now make a start in suggesting tentative values for a safe scale of permissible deflections for current pavement types. It is obvious, of course, that the over-all flexibility of any engineering material will vary with the thickness of the slab or beam, other things being equal. Observations of these deflections and pavement performance seem to justify the strong suspicion that any superior flexibility of present day asphaltic pavements may be due largely to their generally thinner sections and, of course, varies with the amount of asphalt and age. The data seem to raise the question: Are present day asphalt pavements ultimately any more flexible than concrete pavements if constructed to the same thickness and if compared at low temperatures? Subject to many exceptions and individual variations, however, the following scale appears to be a reasonable approximation of values for safe maximum deflections for several types of pavement and base construction:

FIGURE 33

Thickness of Pavement	Type of Pavement	Max. Permissible Deflection for Design Purposes (Tentative)
8-in.	Portland Cement Concrete	0.012-in.
6-in.	Cement Treated Base (Surfaced with Bituminous Pavement)	0.012-in.
4-in.	Asphalt Concrete	0.017-in.
3-in.	Plant Mix on Gravel Base	0.020-in.
2-in.	Plant Mix on Gravel Base	0.025-in.
1-in.	Road Mix on Gravel Base	0.036-in.
1/2-in.	Surface Treatment	0.050-in.

(Bear in mind that the thickness of pavement indicated may or may not be adequate to satisfy the demands of problems one and two as outlined previously.)

This table of tentative deflection values is shown as a curve, Figure 35, and is intended to indicate the safe limits of deflection under some millions of repetitions by the heavy wheel load groups, and as stated previously it is here assumed that for the average highway a mean value would range somewhere in the neighborhood of 15,000-lb. single axle loads. Many instances are known, of course, where the heavy traffic is almost entirely represented by trucks hauling logs, gravel or similar commodities where all axle loads will be near to or frequently exceed the legal load limits.

Figure 34 shows a suggested straight-line relationship on logarithmic paper between pavement thickness and permissible deflection.

In view of the fact that the majority of the deflections indicate a linear relationship between deflection and magnitude of the axle or wheel loads, a chart may be constructed, Figure 35, showing the relative deflections which would be developed under a range of loads, the lines being drawn through points above the 15,000-lb. axle load corresponding to the various maximum deflections suggested in Table Figure 33 and Chart 34.

TANDEM AXLES VERSUS SINGLE AXLES

Among interesting facts brought to light by these field measurements of deflections is the evidence of a variable but apparently orderly relationship between the deflections resulting from single axle loads compared to those caused by loads placed in the close proximity that occurs with tandem axles.

In the majority of pavements that were studied the deflection measurements have been recorded for both single and tandem axle loads. Figure 36 shows the rapid reversal of pavement bending under a 31,000-lb. tandem axle on a section of badly cracked asphaltic pavement illustrated in Figure 6, and for comparison the pattern registered by a single axle 18,000-lb. load. Figure 37 shows the deflections under both a single axle and tandem axles on the excellent pavement supported by a cement treated base shown previously by Figure 17. Figure 38 illustrates the deflections of a concrete pavement under a single axle load and also the deflections of the same slab under tandem axles.

For bituminous pavements on gravel bases the foregoing indicates that when two axle loads are closely spaced as in the case of tandem axles then each trip of a truck produces a repetition of load for each axle regardless of spacing. For concrete pavement, on the other hand, there is little or no rebound between such closely spaced axles, and, therefore, a tandem axle should be counted as one axle load for purposes of summarizing the effects of traffic in equivalent wheel load (EWL) computations, See Figure 39 for comparisons, for instance. The differences between the effects of single axle and tandem axle loads are further evident when the total amount of pavement deflection is compared. Figure 40 illustrates that an 18,000-lb. single axle and a 32,000-lb. tandem axle produce almost exactly the same deflections for depths up to nine feet. This is the same pavement referred to by Figure 16. This same close correspondence has been noted on most of the pavements consisting of a bituminous surface over a granular base. Figure 41 shows that where a cement treated base exists there is a small, but consistent difference, the semi-rigid base showing greater deflection under a tandem representing two 16,000-lb. axle loads than for a single 18,000-lb. Finally, Figure 42 shows the marked difference where a concrete pavement is involved. Here the deflection under a 32,000-lb. tandem axle is very much greater than for the 18,000-lb. single.

Thus it appears that the deflections of bituminous pavements over a gravel base subjected to a 32,000-lb. load on tandem axles show an average value almost identical to that produced by a 19,000-lb. single axle. Figure 43 shows the close and consistent relationship between all the available deflection values on bituminous pavements over gravel bases for both tandem and single axles at these loads. Similarly, Figure 44 shows that a 24,000-lb. tandem is equal to a 13,000 single axle. However, the relationship over a cement treated base is indicated in Figure 45 as 32,000-lb. tandem = 21,000-lb. single axle and for portland cement concrete Figure 46, 32,000-lb. tandem = 24,000-lb. single. This last relationship seems to be rather strikingly confirmed by an examination of the data for test road One-MD -- where the destructive effect as indicated by the lineal feet of cracking produced by the 32,000-lb. tandem axles appears to be almost exactly equal to the amount that would be indicated by extrapolation for the same number of trips of a 24,000-lb. single axle load. Extrapolating the trend described above means that an analysis of strength requirements for a bridge deck should show still less difference between a load carried on single axles as compared to tandem axles. The relationship will vary with the length of the span, but according to the AASHO formula for a 32-ft. span a 32,000-lb. tandem should be equal to a 28,000-lb. single axle load.

The inter-relationship between load distribution and pavement strength may be illustrated graphically by means of a chart, Figure 47. This chart indicates the "safe" deflections for several types of pavement and the relative deflection that would result for any condition from any change in load either single axle or tandem. The effect of slab strength is evident. This chart is an attempt to rationalize the data where the deflections vary directly with load and indicates the more or less orderly relationship between magnitude of load, axle spacing, pavement type and deflections.

Mention was made earlier of the concept of stiffness and reference was made to work with the Shell vibration machine. In closing it should be mentioned that there appears to be some correlation between evaluations tentatively established by Nijboer and associates for pavements in Europe and the indications derived from deflection measurements in California. As outlined in the footnote the concept of stiffness as used by Nijboer covers the total resistance of the pavement, base and soil. This is also true of the deflection measurements as reported herein. Therefore, the relationship between stiffness and deflection may be expressed by the formula

$$S = \frac{L}{d}$$

Where S = the stiffness in kip per inch

d = the deflection in inches

L = the wheel load = $\frac{\text{axle load}}{2}$

Figure 48 is a chart showing the relationship between axle loads, deflections and computed stiffness. As stated previously and indicated in Figure 33, we have tentatively assigned limits for the deflection that pavements of various thickness and type will safely withstand over a period of years. These values, of course, are only tentative at this time but it appears that the heavier pavements should be limited somewhere between 0.012-in. and 0.020-in. Referring to Chart 48, this range of deflections is the equivalent of a stiffness factor in the approximate range of 400 to 600 kip per inch. For comparison, the limiting value of stiffness suggested by Nijboer and van der Poel(2) ranges from 570 to 1140 kip per inch. This agreement is not too close but the comparison is offered primarily to indicate that our present ideas of a limiting deflection are liberal rather than otherwise and if we accept Nijboer's conclusions permissible deflections would range between 0.006-in. and 0.013-in.

CONCLUSIONS

In conclusion then it may be stated that there is an unusually close correlation between observations of cracking and fatigue type failures in bituminous pavements and the measured deflections which the pavement must undergo with each passing wheel load. These deflections appear to be associated with compression and rebound in the soil, and it is obvious that most pavements will withstand a few such deformations if not too often repeated. It may be said that a principal destructive force is the energy stored in the subgrade by each passing wheel.

It appears that the problem has three possible solutions as follows:

(1) provide a pavement* or wearing surface layer that is sufficiently flexible to accommodate repeated substantial vertical deflections without serious cracking.

(2) decrease the magnitude of the vertical deflections to a tolerable limit by providing greater stiffness by means of greater depths of granular bases and subbases under the pavement, or

(3) provide a pavement with a slab strength sufficient to sustain the forces induced by traffic which cause cracking.

*The term "pavement" or surface course as used herein, includes that portion of the overall pavement structure lying on the base and which is generally a mixture of aggregate and asphalt or portland cement.

With regard to (1) above, the only method of utilizing flexibility, at present, is to use a thin bituminous surface treatment as there are no materials available today at reasonable cost to construct truly flexible surfaces of substantial thickness. Such thin "pavements" are, of course, vulnerable to other destructive effects of heavy traffic and to adverse weather conditions. Also, a thin surface is not able to provide the necessary strength or weight to carry loads over cohesionless sands or over plastic soils.

Solution (2) above is in recognition of the fact that the magnitude of the deflections are related to the overall pavement structure thickness and that actual deflections can be reduced to an acceptable limit simply by increasing the thickness of a non-resilient base and/or subbase. At present, pavement and base thickness design procedures are predicated primarily on ability to carry loads over plastic soils (i.e. on resistance values and expansion pressures), which means providing sufficient cover thickness to support traffic over soils of low bearing or resistance value. It appears that it will now be necessary to develop a second pavement structural design procedure based on resiliency factors and fatigue susceptibility in which soil resilience, magnitude of loads, load repetition and the stiffness of the pavement and base are all related in order to provide an adequate design. Both procedures will then have to be considered and the thickest of the two pavement sections selected. It appears to be true, fortunately, that in many cases (perhaps in a majority of instances) low resistance values and high resilience characteristics go hand-in-hand, and consequently, most pavements designed on strength factors are adequate from the standpoint of resiliency effects. However, there is evidence that this is not always the case and there does exist an element of doubt which should not be allowed to exist if it can be eliminated.

The third approach to the problem, as given in (3) above, calls for a pavement of considerable slab strength, such as heavy portland cement concrete slabs or the use of cement treated bases in conjunction with a substantial thickness (4" minimum) of asphalt pavement. It has been observed, and substantiated to a large extent by the data included in this paper, that pavements or pavement structures of high slab strength need not be as great in overall thickness as sections utilizing lower slab strengths in order to perform satisfactorily in carrying traffic over resilient soils. However, for modern industrial traffic all sections must still be of substantial thickness.

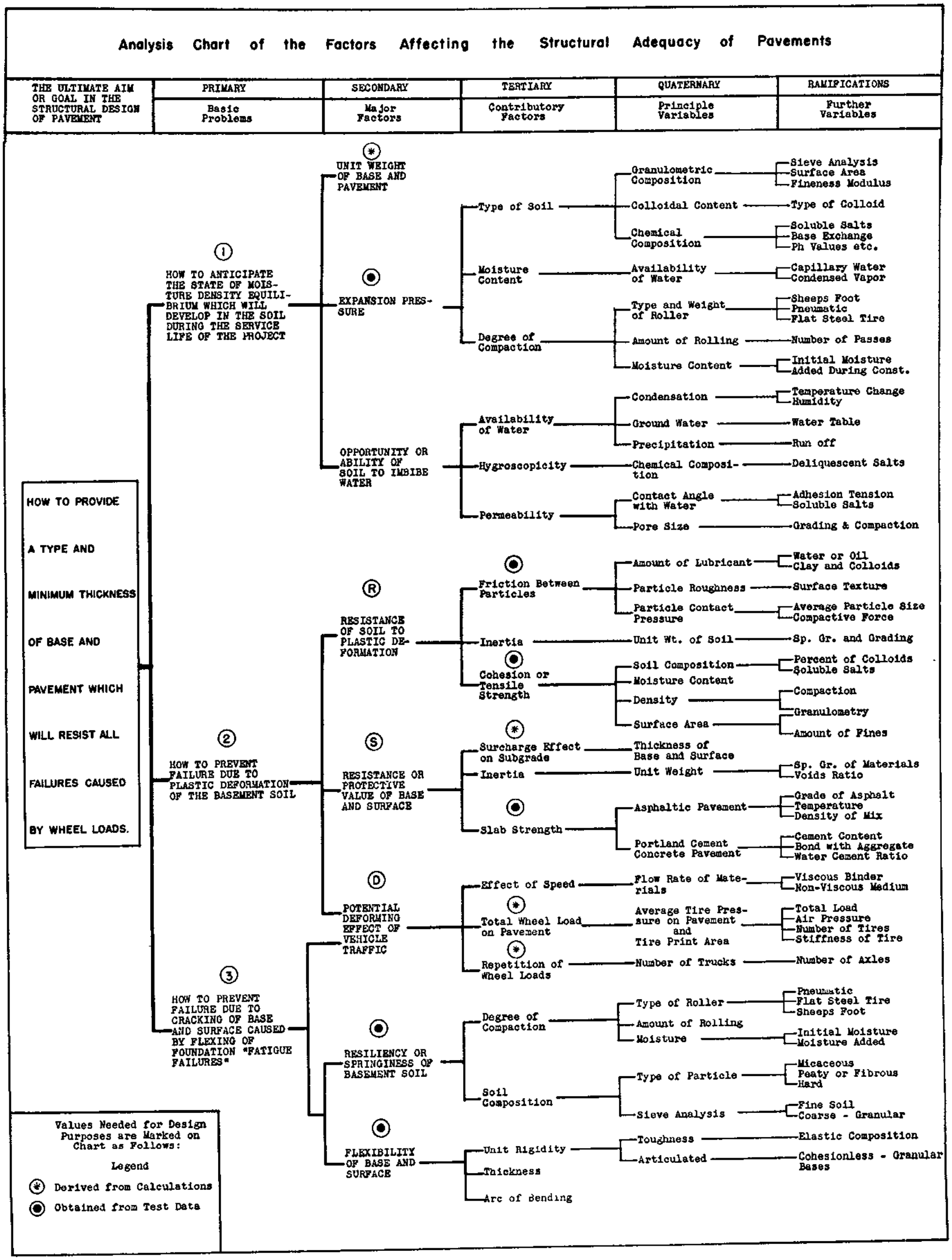
It appears that the engineer is faced with the necessity of designing pavements of the various types described above in order to meet all three primary problems; namely, potential expansion, plastic deformation and finally the resilience of the underlying materials.

After designing comparable sections which will satisfy the above structural and physical requirements it is then the engineer's responsibility to make an economic analysis to determine which one should be specified for a given location.

None of the foregoing promises to make life any simpler for those who must design highway or airport pavements.

ACKNOWLEDGMENT

The foregoing represents selected examples and extracts from data obtained in an extensive investigation which has been under way for over four years. It is difficult to list by name all of those who have been involved and who have contributed time and effort to the various phases of this work. Most of the studies have been carried on in the Pavement Section of this Laboratory which is under the direction of Ernest Zube. The principal task of correlating and assembling the data, analyzing and studying the results has been under the direction of George Sherman, assisted by Robert Bridges. The cutting of cores, securing soil samples and installing gauge units were handled by Charles Clawson. James E. Barton and assistants were responsible for the General Electric travel gauge, electronic equipment and recording of the deflections. I should like to acknowledge the courtesy and assistance of the Shell Oil Company who made available the Shell Company vibrator and the services of a skilled operator in order to compare "dynamic" deflections and other measurements without the necessity for constructing such an expensive unit. The author has appreciated helpful discussions with Mr. W. N. Carey, Jr. of the Highway Research Board and Mr. A. C. Benkelman of the Bureau of Public Roads.



Analysis Chart of the Factors Affecting the Structural Adequacy of Pavements
THE ULTIMATE AIM OR GOAL IN THE STRUCTURAL DESIGN OF PAVEMENT
PRIMARY
Basic Problems
SECONDARY
Major Factors
TERTIARY
Contributory Factors
QUATERNARY
Principle Variables
RAMIFICATIONS
Further Variables
HOW TO PROVIDE A TYPE AND MINIMUM THICKNESS OF BASE AND PAVEMENT WHICH WILL RESIST ALL FAILURES CAUSED BY WHEEL LOADS.
1
HOW TO ANTICIPATE THE STATE OF MOISTURE DENSITY EQUILIBRIUM WHICH WILL DEVELOP IN THE SOIL DURING THE SERVICE LIFE OF THE PROJECT
UNIT WEIGHT OF BASE AND PAVEMENT
EXPANSION PRESSURE
Type of Soil
Granulometric Composition
Sieve Analysis
Surface Area
Fineness Modulus
Colloidal Content
Type of Colloid
Chemical Composition
Soluble Salts
Base Exchange
Ph Values etc.
Moisture Content
Availability of Water
Capillary Water
Condensed Vapor
Degree of Compaction
Type and Weight of Roller
Sheeps Foot
Pneumatic
Flat Steel Tire
Amount of Rolling
Number of Passes
Moisture Content
Initial Moisture
Added During Const.
OPPORTUNITY OR ABILITY OF SOIL TO IMBIBE WATER
Availability of Water
Condensation
Temperature Change
Humidity
Ground Water
Water Table
Precipitation
Run off
Hygroscopicity
Chemical Composition
Deliquescent Salts
Permeability
Contact Angle with Water
Adhesion Tension
Soluble Salts
Pore Size
Grading & Compaction
2
HOW TO PREVENT FAILURE DUE TO PLASTIC DEFORMATION OF THE BASEMENT SOIL
R
RESISTANCE OF SOIL TO PLASTIC DEFORMATION
Friction Between Particles
Amount of Lubricant
Water or Oil
Clay and Colloids
Particle Roughness
Surface Texture
Particle Contact Pressure
Average Particle Size
Compactive Force
Inertia
Unit Wt. of Soil
Sp. Gr. and Grading
Cohesion or Tensile Strength
Soil Composition
Percent of Colloids
Soluble Salts
Moisture Content
Density
Compaction
Granulometry
Surface Area
Amount of Fines
S
RESISTANCE OR PROTECTIVE VALUE OF BASE AND SURFACE
Surcharge Effect on Subgrade
Thickness of Base and Surface
Inertia
Unit Weight
Sp. Gr. of Materials
Voids Ratio
Slab Strength
Asphaltic Pavement
Grade of Asphalt
Temperature
Density of Mix
Portland Cement Concrete Pavement
Cement Content
Bond with Aggregate
Water Cement Ratio
D
POTENTIAL DEFORMING EFFECT OF VEHICLE TRAFFIC
Effect of Speed
Flow Rate of Materials
Viscous Binder
Non-Viscous Medium
Total Wheel Load on Pavement
Average Tire Pressure on Pavement and Tire Print Area
Total Load
Air Pressure
Number of Tires
Stiffness of Tire
Repetition of Wheel Loads
Number of Trucks
Number of Axles
3
HOW TO PREVENT FAILURE DUE TO CRACKING OF BASE AND SURFACE CAUSED BY FLEXING OF FOUNDATION "FATIGUE FAILURES"
RESILIENCY OR SPRINGINESS OF BASEMENT SOIL
Degree of Compaction
Type of Roller
Pneumatic
Flat Steel Tire
Sheeps Foot
Amount of Rolling
Moisture
Initial Moisture
Moisture Added
Soil Composition
Type of Particle
Micaceous
Peaty or Fibrous
Hard
Sieve Analysis
Fine Soil
Coarse - Granular
FLEXIBILITY OF BASE AND SURFACE
Unit Rigidity
Toughness
Elastic Composition
Articulated
Cohesionless - Granular Bases
Thickness
Arc of Bending
Values Needed for Design Purposes are Marked on Chart as Follows:
Legend
Derived from Calculations
Obtained from Test Data

FIG. 1

Relationship between Fundamental Factors Governing the Structural Design of Pavements and Bases

Properties of pavements and subgrades susceptible to measurement which must be taken into account or manipulated in the process of designing adequate pavements and bases		In order to select the type and thickness of pavement and base, three distinct problems must be considered and solved			
		1 EQUILIBRIUM How to maintain equilibrium of moisture and density in basement soil by restraining expansion	2 DISTORTION How to prevent plastic deformation of the basement soil under heavy wheel loads	3 REBOUND How to prevent subgrade rebound from destroying the pavement through fatigue	
Properties	**Test Method**				
Basement Soil					Factors which must be evaluated to determine the problem
A. Internal Friction	Stabilometer		A. (Major)		
B. Cohesion	Cohesiometer		B. (Minor)	B. (Minor)	
C. Swell Pressure	Swell Dynamometer	C. (Major)	C		
D. Resilience	Resiliometer			D. (Major)	
Pavements & Bases					Factors which may be adjusted or manipulated in solving the problem
a. Flexural Strength	Cohesiometer Beam Tests		a	a	
b. Weight	Thickness Specific Gravity	b	b	b	
c. Flexibility or lack of Brittleness	Test Method being developed			c	

Figure 2

ANALYSIS CHART ILLUSTRATING TYPES OF PAVEMENT DEFORMATION

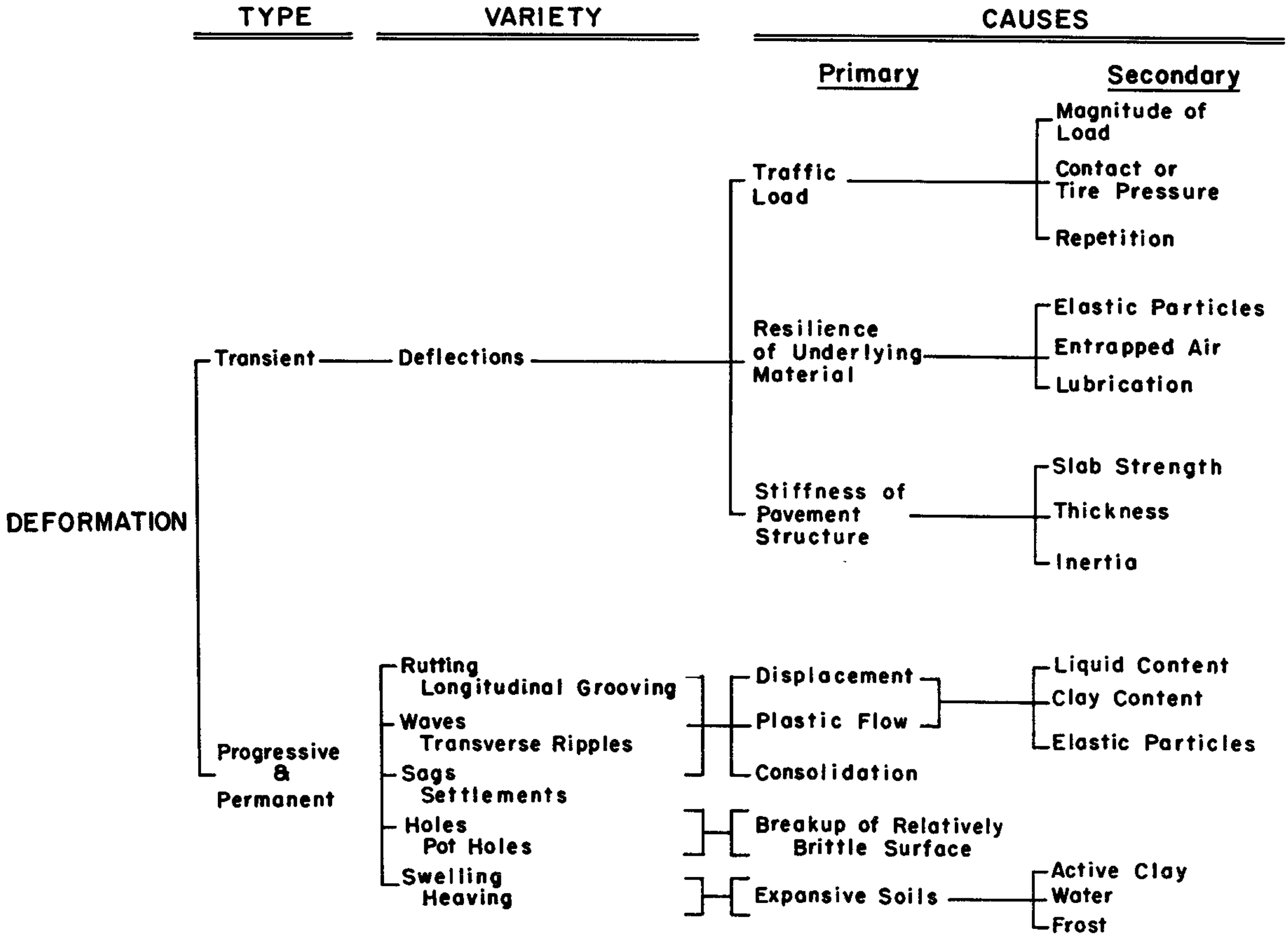
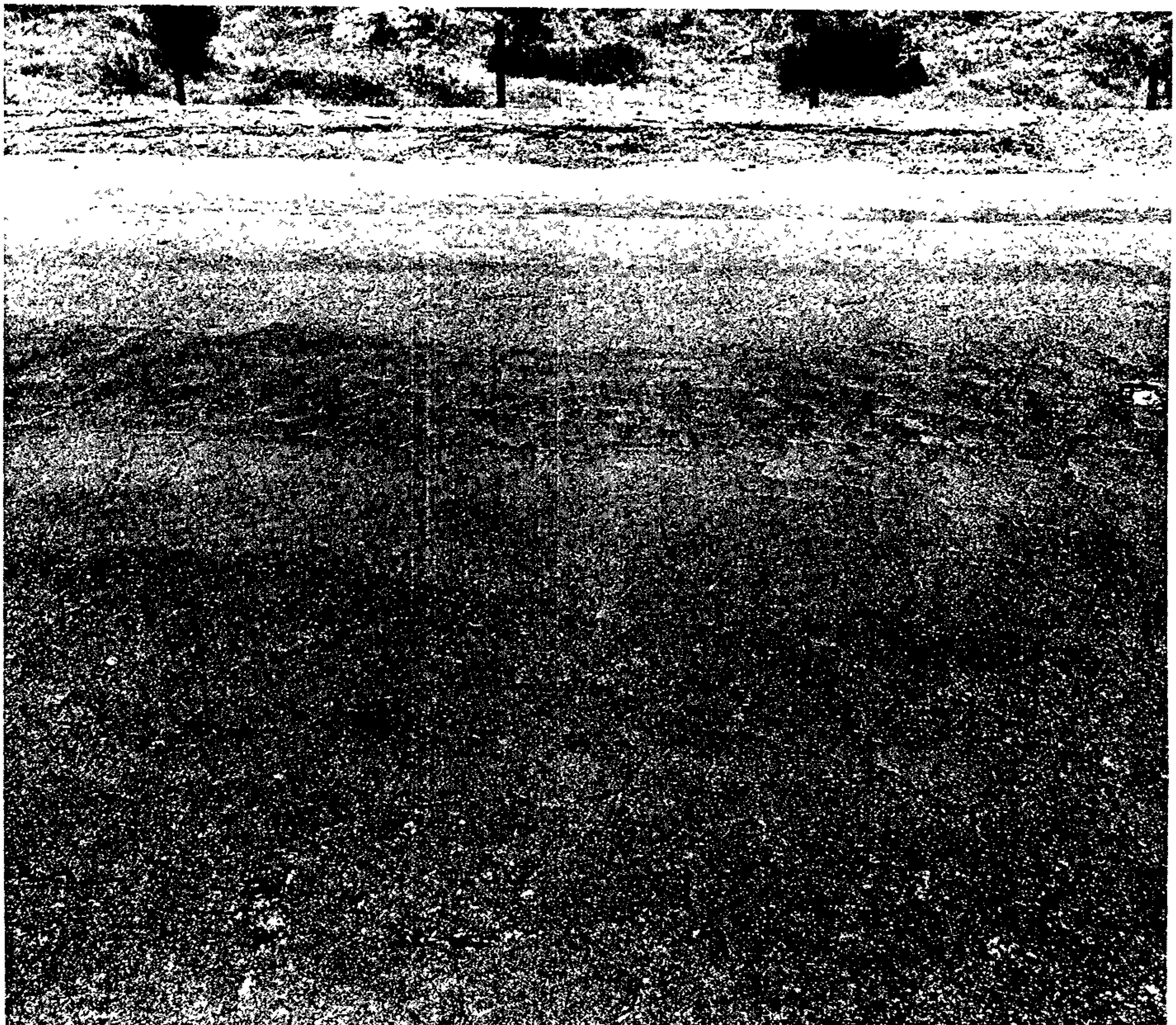


FIG. 3



Typical Illustration of
"Chicken Wire" or "Alligator" Cracking

Fig. 4

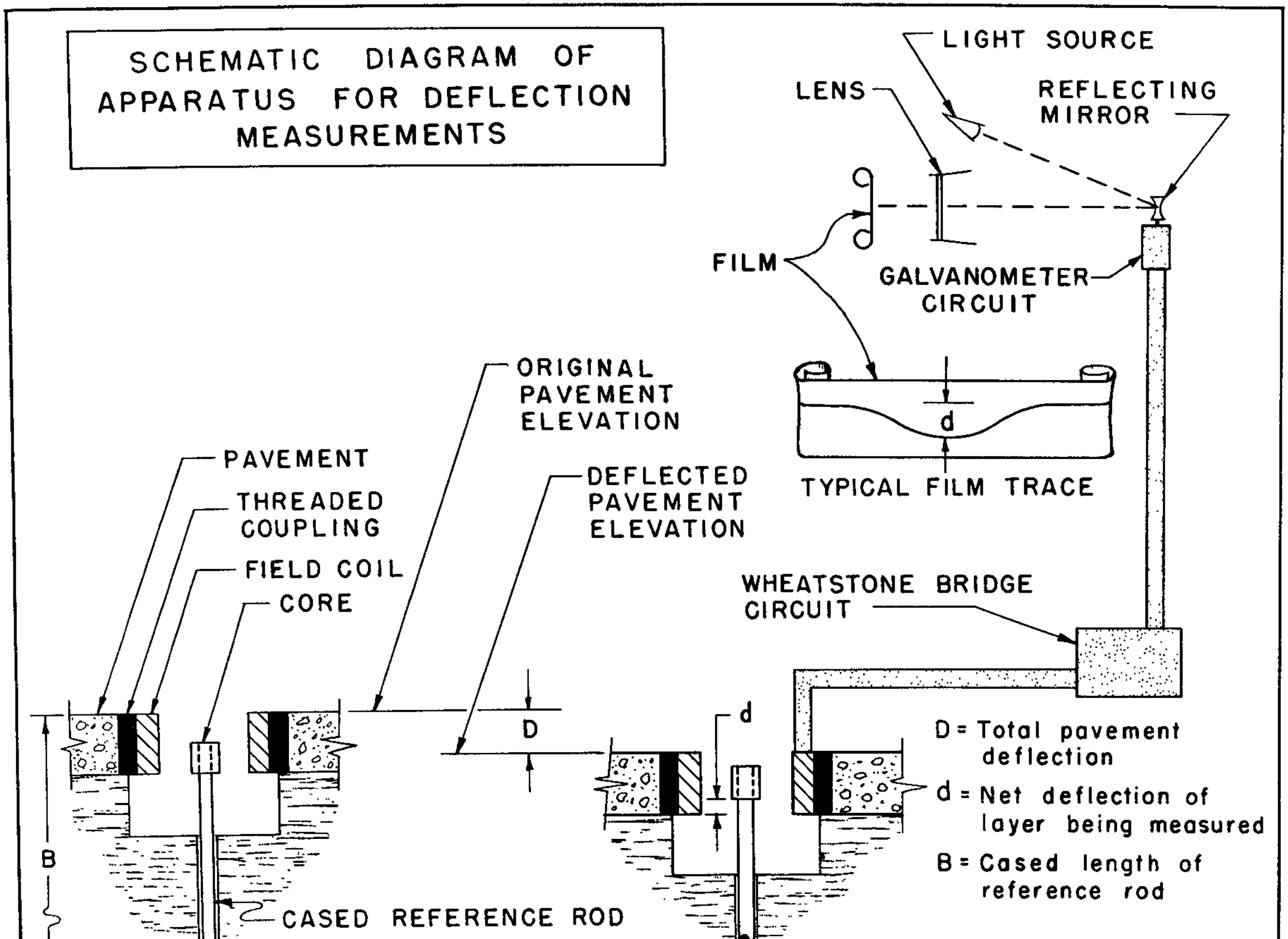

SCHEMATIC DIAGRAM OF APPARATUS FOR DEFLECTION MEASUREMENTS
LIGHT SOURCE
LENS
REFLECTING MIRROR
FILM
GALVANOMETER CIRCUIT
d
TYPICAL FILM TRACE
ORIGINAL PAVEMENT ELEVATION
PAVEMENT
THREADED COUPLING
FIELD COIL
CORE
DEFLECTED PAVEMENT ELEVATION
WHEATSTONE BRIDGE CIRCUIT
D
d
D = Total pavement deflection
d = Net deflection of layer being measured
B = Cased length of reference rod
B
CASED REFERENCE ROD

Fig. 5

FIG. 6

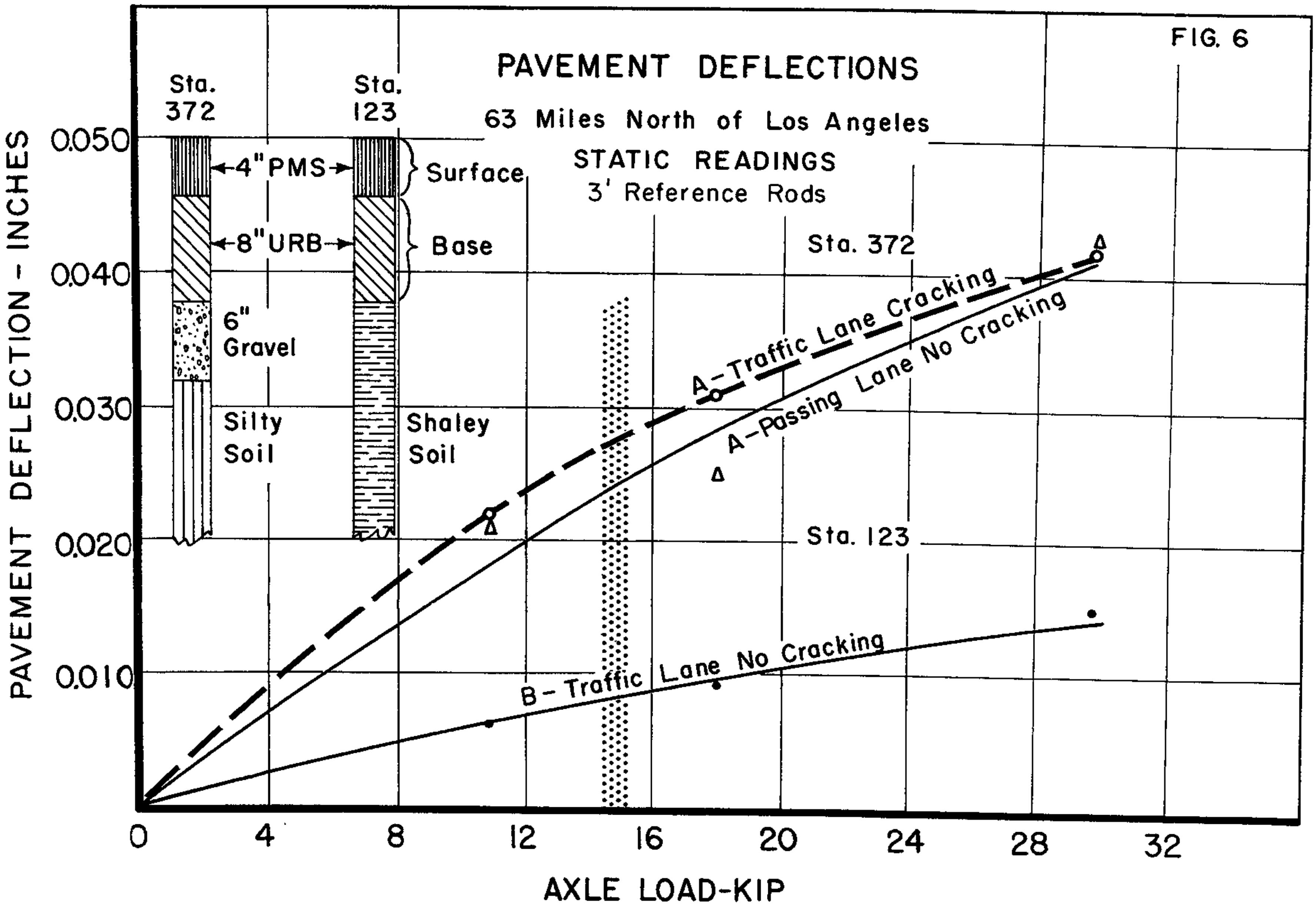
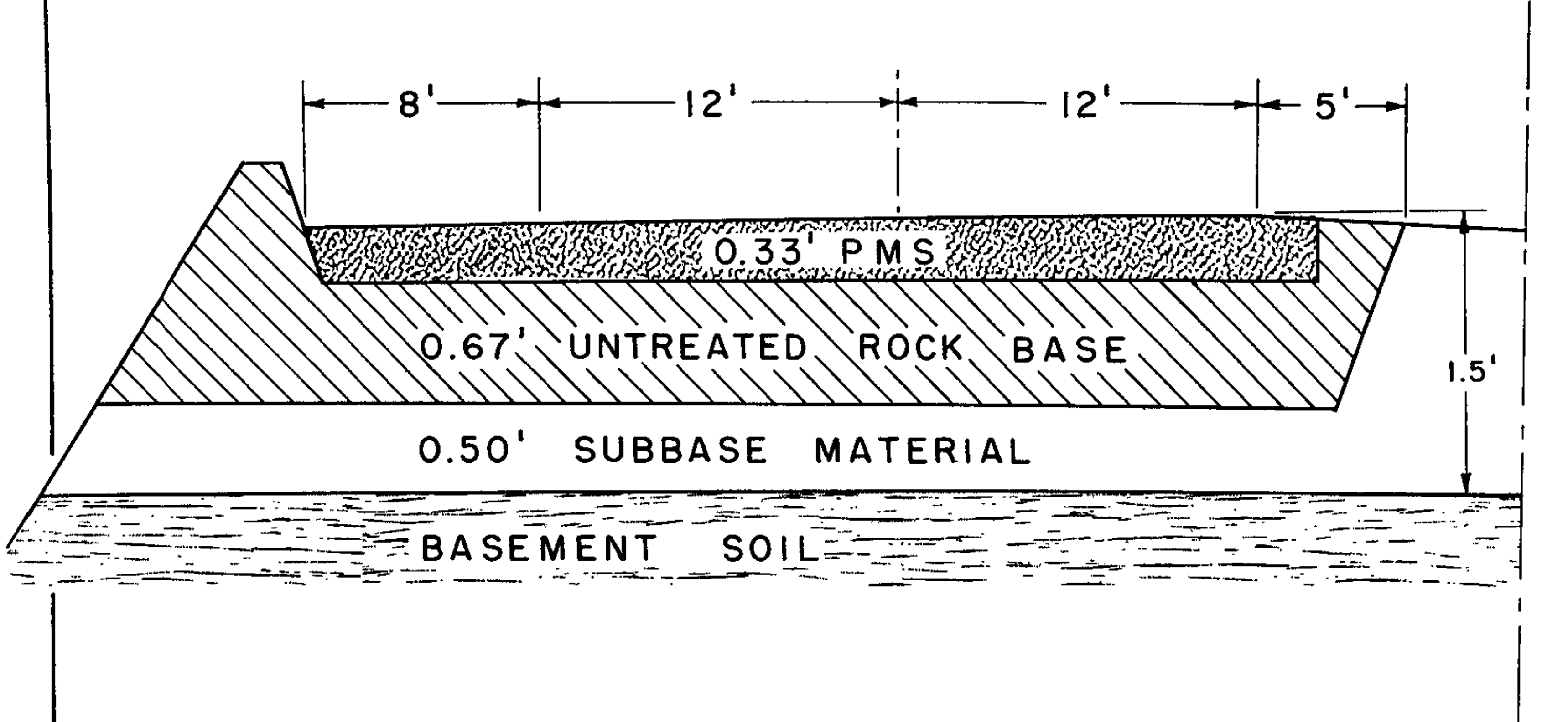

PAVEMENT DEFLECTIONS
63 Miles North of Los Angeles
STATIC READINGS
3' Reference Rods
Sta. 372
Sta. 123
4"PMS
8"URB
6" Gravel
Silty Soil
Surface
Base
Shaley Soil
Sta. 372
Sta. 123
A-Traffic Lane Cracking
A-Passing Lane No Cracking
B-Traffic Lane No Cracking
PAVEMENT DEFLECTION - INCHES
0.050
0.040
0.030
0.020
0.010
0
4
8
12
16
20
24
28
32
AXLE LOAD-KIP

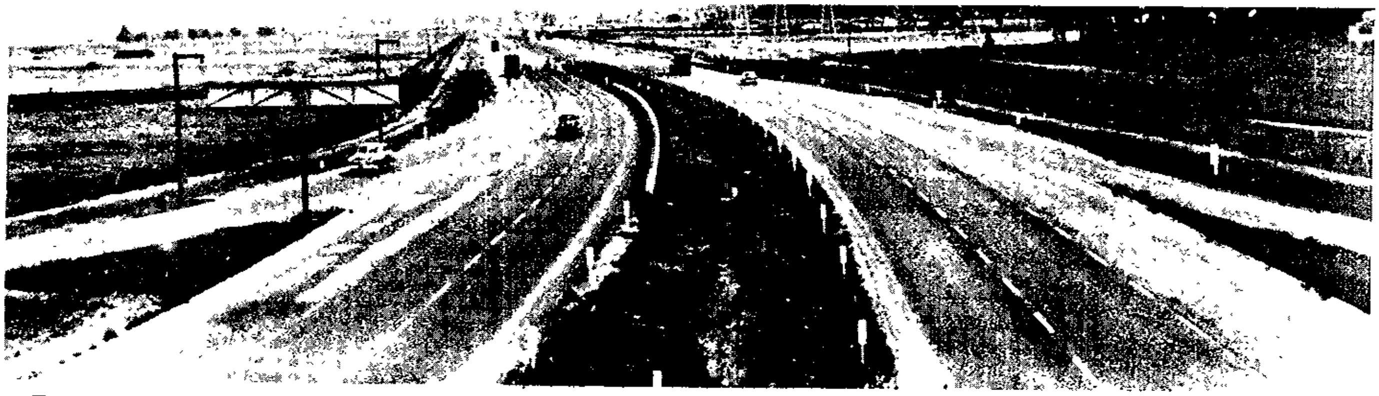
Fig. 7

TYPICAL CROSS SECTION

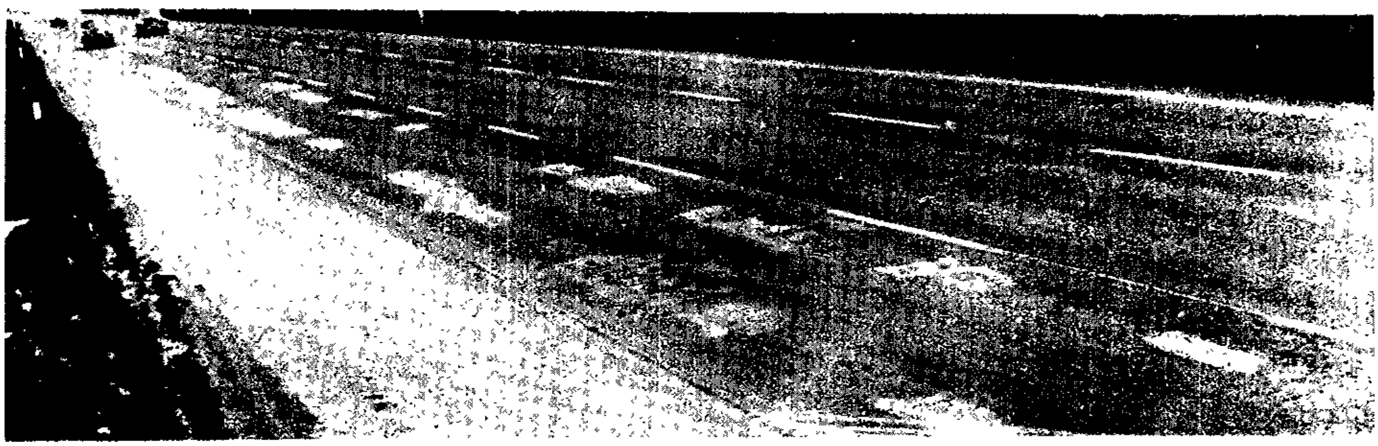
ROAD VII - LA - 4 - J

STA. 372+20 PAVEMENT CRACKED





Sta. 395± General Outside Lane Failure



Sta. 498± General Outside Lane Failure

Typical Failed Areas
San Francisco Bayshore Freeway

Fig. 8

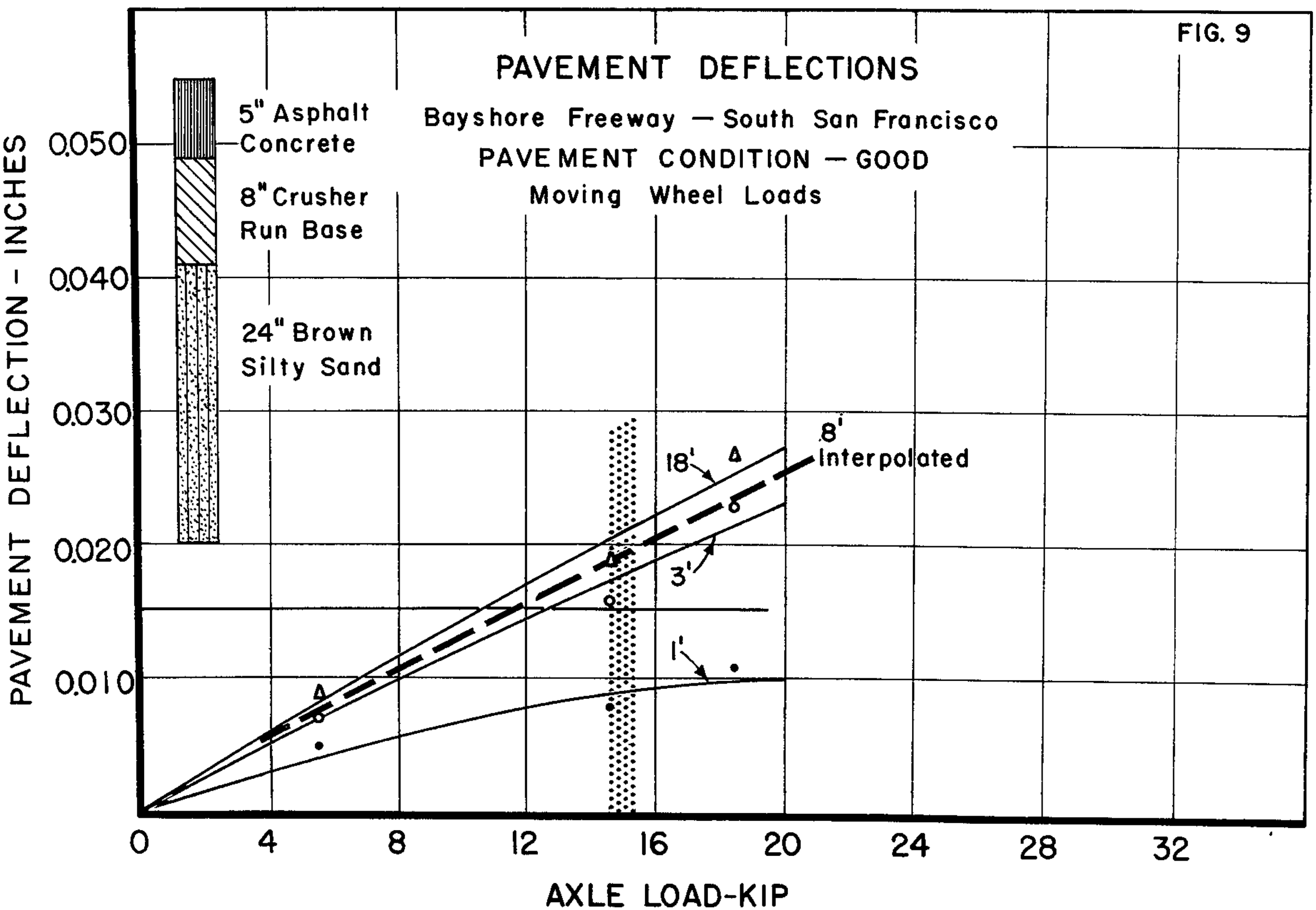


FIG. 9
PAVEMENT DEFLECTIONS
Bayshore Freeway — South San Francisco
PAVEMENT CONDITION — GOOD
Moving Wheel Loads
5" Asphalt Concrete
8" Crusher Run Base
24" Brown Silty Sand
18'
8' Interpolated
3'
1'
PAVEMENT DEFLECTION - INCHES
0.050
0.040
0.030
0.020
0.010
0
4
8
12
16
20
24
28
32
AXLE LOAD-KIP

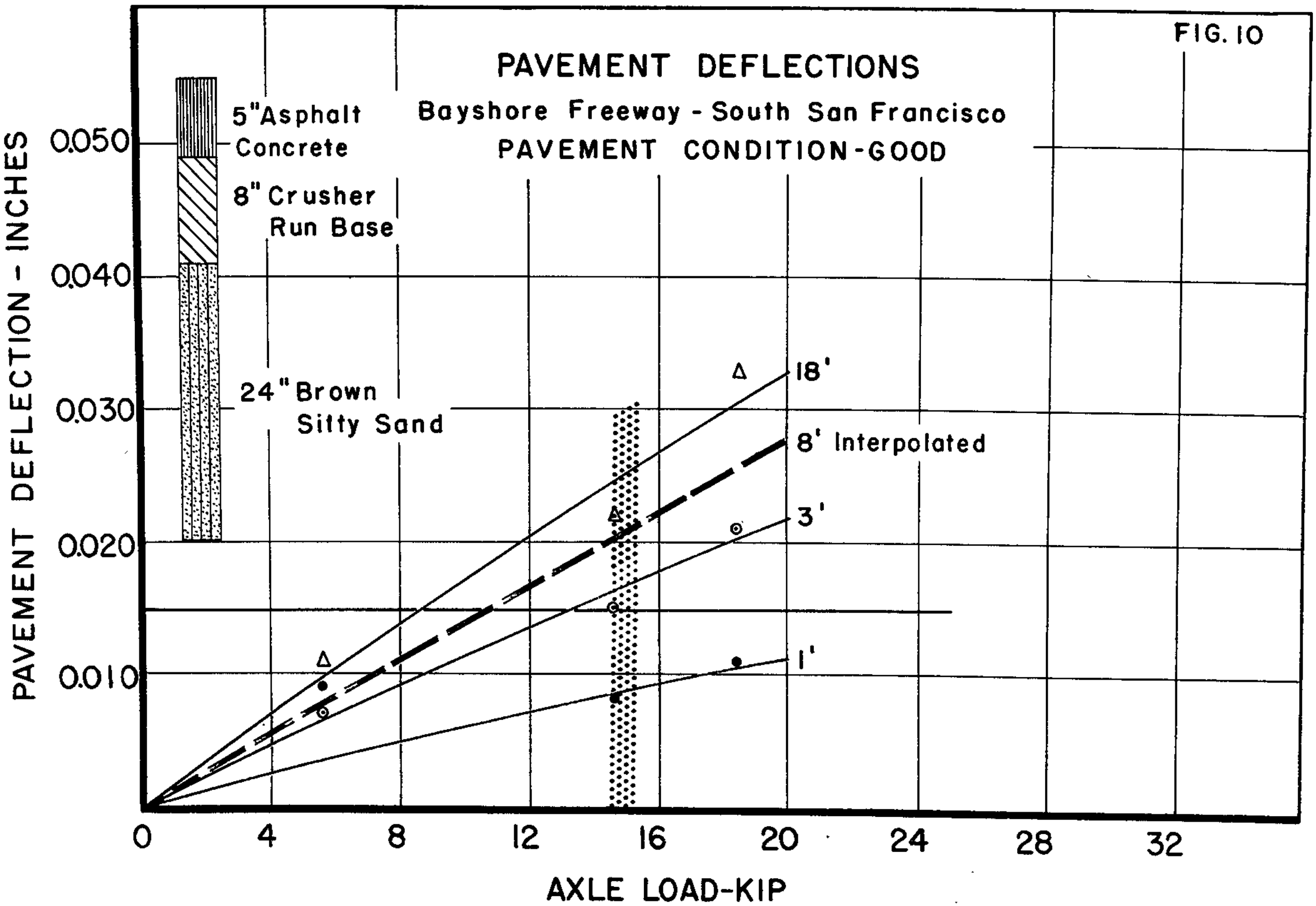
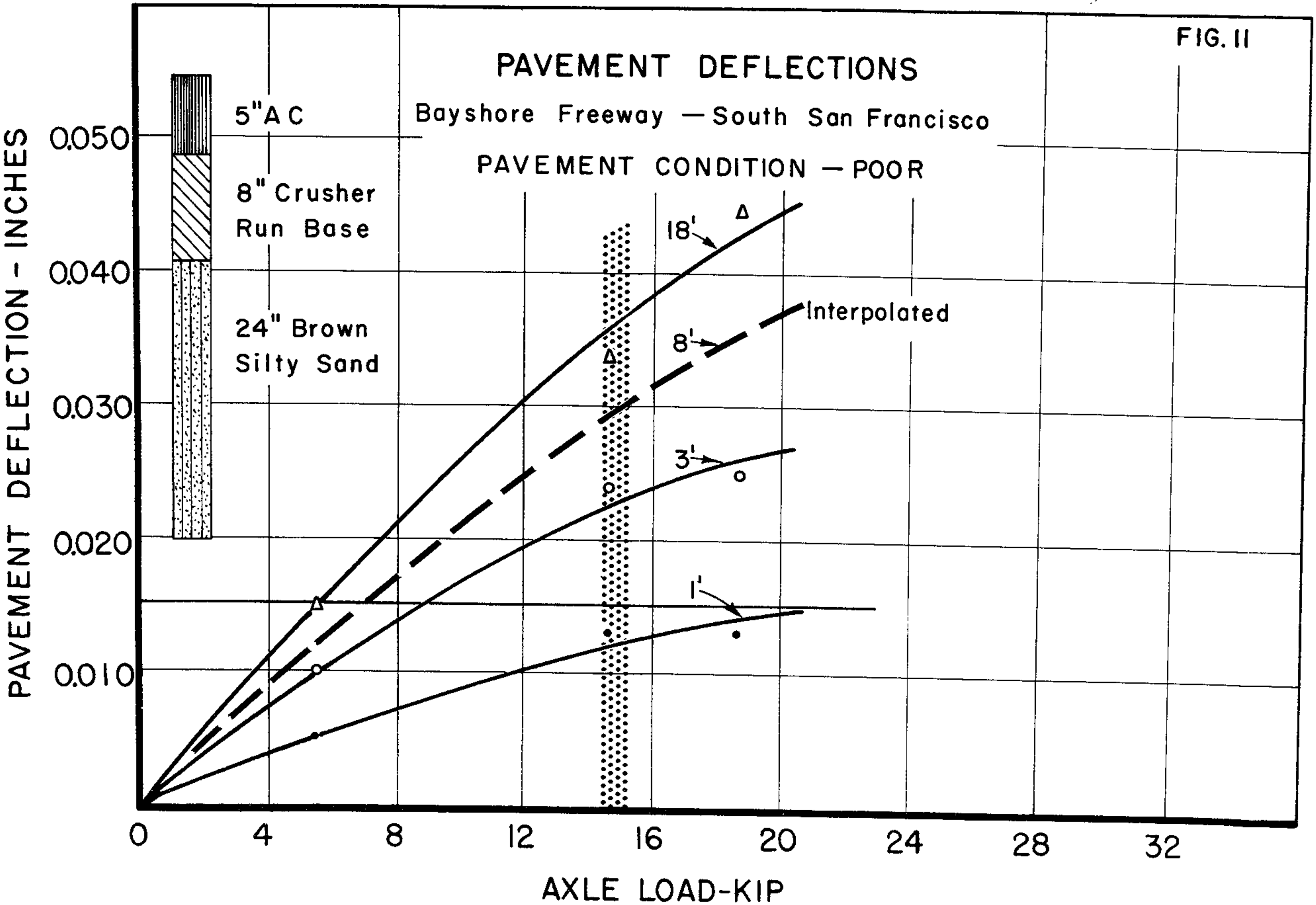


FIG. 10
PAVEMENT DEFLECTIONS
Bayshore Freeway - South San Francisco
PAVEMENT CONDITION-GOOD
5"Asphalt Concrete
8" Crusher Run Base
24" Brown Silty Sand
18'
8' Interpolated
3'
1'
PAVEMENT DEFLECTION - INCHES
0.050
0.040
0.030
0.020
0.010
0
4
8
12
16
20
24
28
32
AXLE LOAD-KIP

FIG. 11


PAVEMENT DEFLECTIONS
Bayshore Freeway — South San Francisco
PAVEMENT CONDITION — POOR
5" A C
8" Crusher Run Base
24" Brown Silty Sand
18'
8'
Interpolated
3'
1'
0.050
0.040
0.030
0.020
0.010
PAVEMENT DEFLECTION - INCHES
0
4
8
12
16
20
24
28
32
AXLE LOAD-KIP

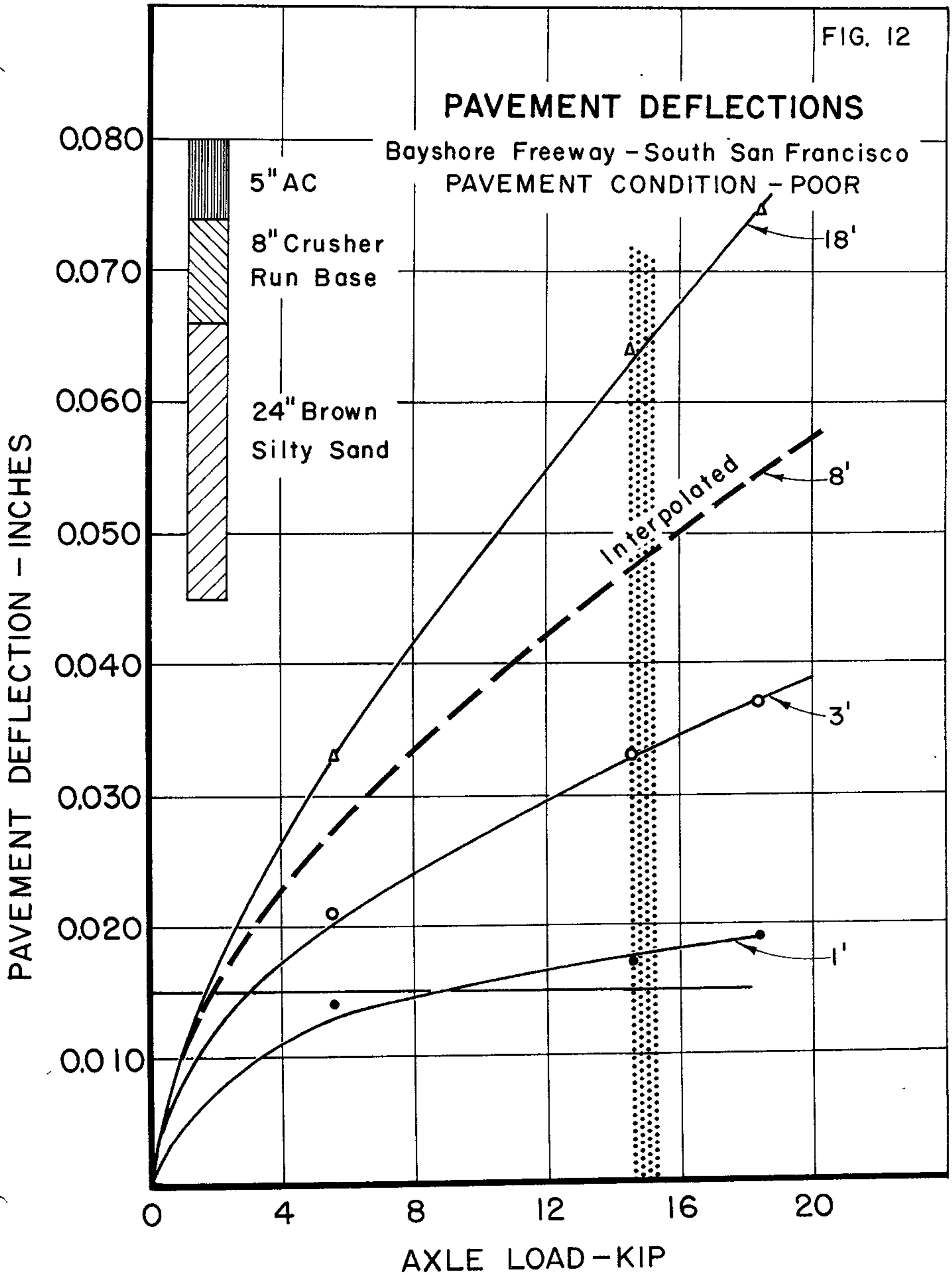


FIG. 12
PAVEMENT DEFLECTIONS
Bayshore Freeway - South San Francisco
PAVEMENT CONDITION - POOR
5" AC
8" Crusher Run Base
24" Brown Silty Sand
Interpolated
18'
8'
3'
1'
0.080
0.070
0.060
0.050
0.040
0.030
0.020
0.010
PAVEMENT DEFLECTION - INCHES
0
4
8
12
16
20
AXLE LOAD - KIP

FIG. 13

PAVEMENT DEFLECTIONS

BAYSHORE FREEWAY - SOUTH SAN FRANCISCO

14000 lb. - Wheel Load

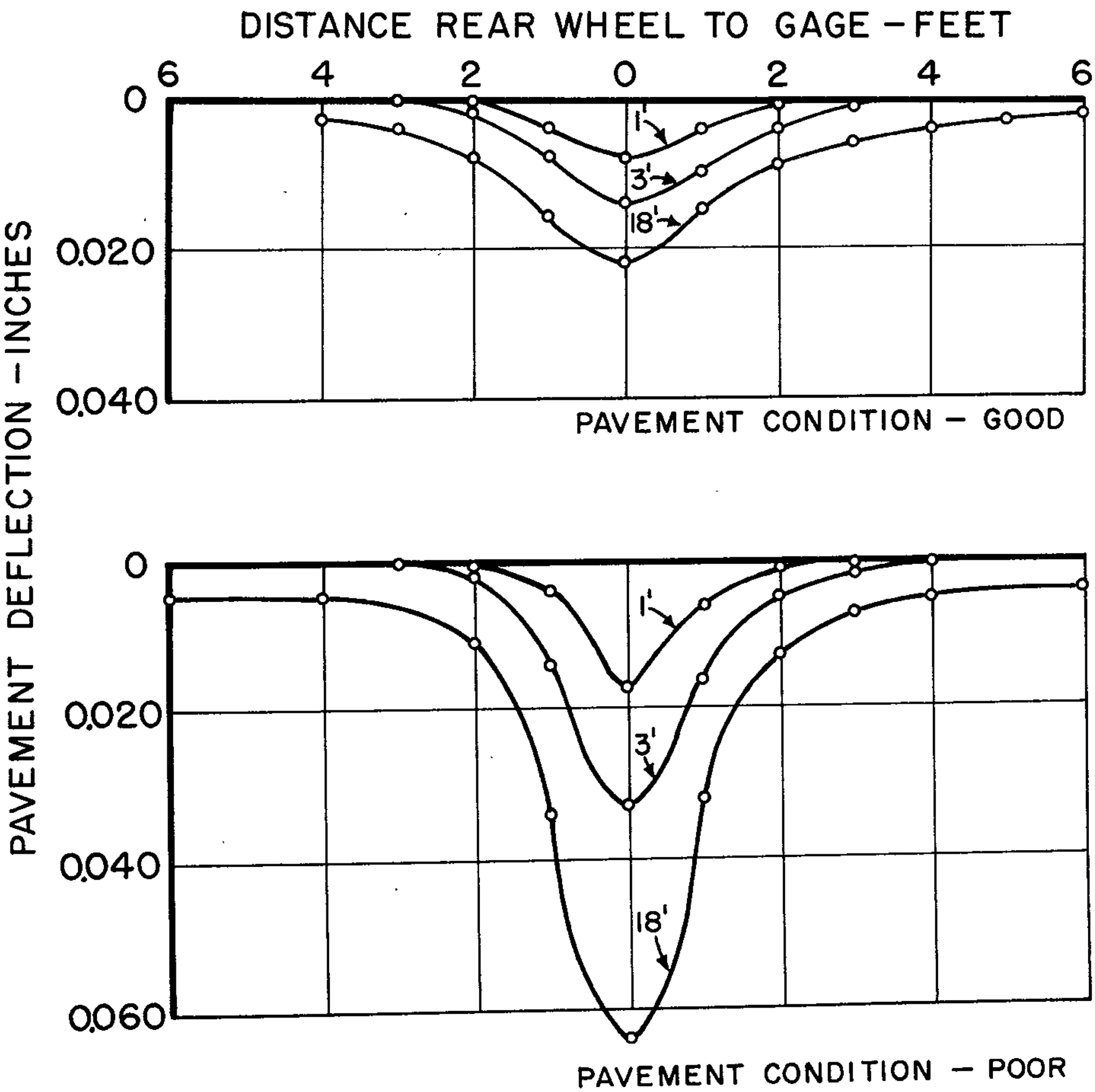


FIG. 14

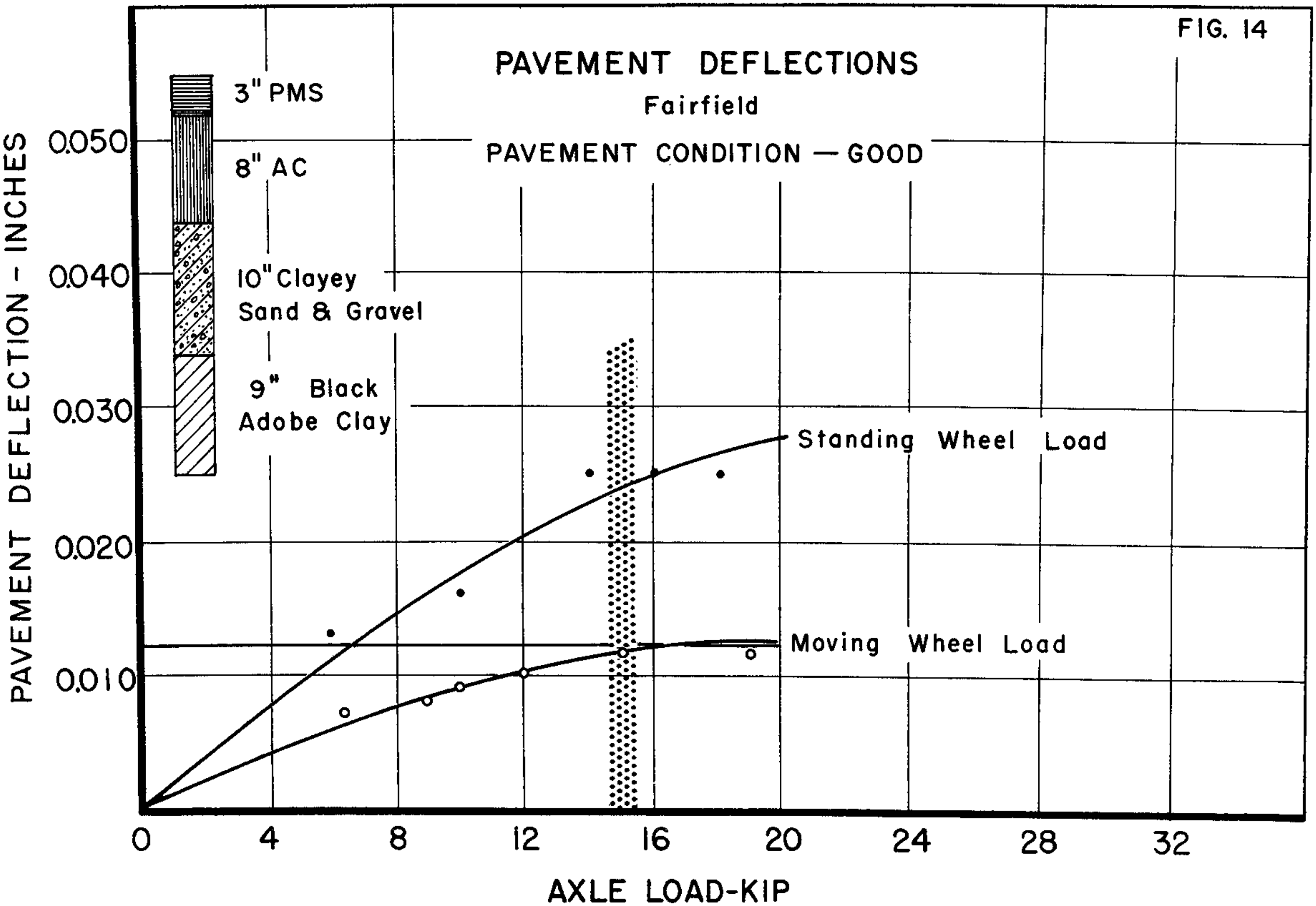

PAVEMENT DEFLECTIONS
Fairfield
PAVEMENT CONDITION — GOOD
3" PMS
8" AC
10" Clayey Sand & Gravel
9" Black Adobe Clay
Standing Wheel Load
Moving Wheel Load
PAVEMENT DEFLECTION - INCHES
0.050
0.040
0.030
0.020
0.010
0
4
8
12
16
20
24
28
32
AXLE LOAD-KIP

FIG. 15

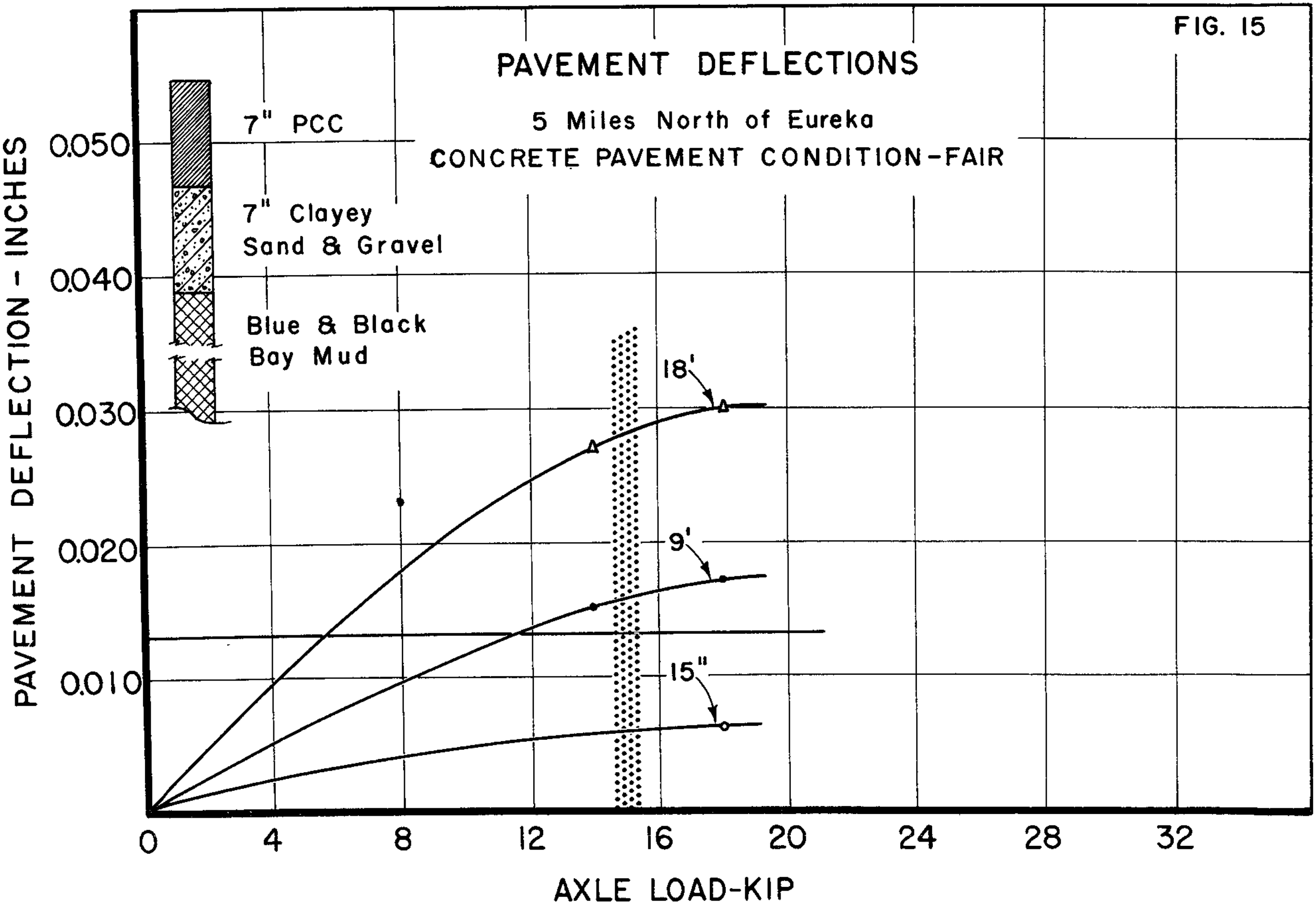
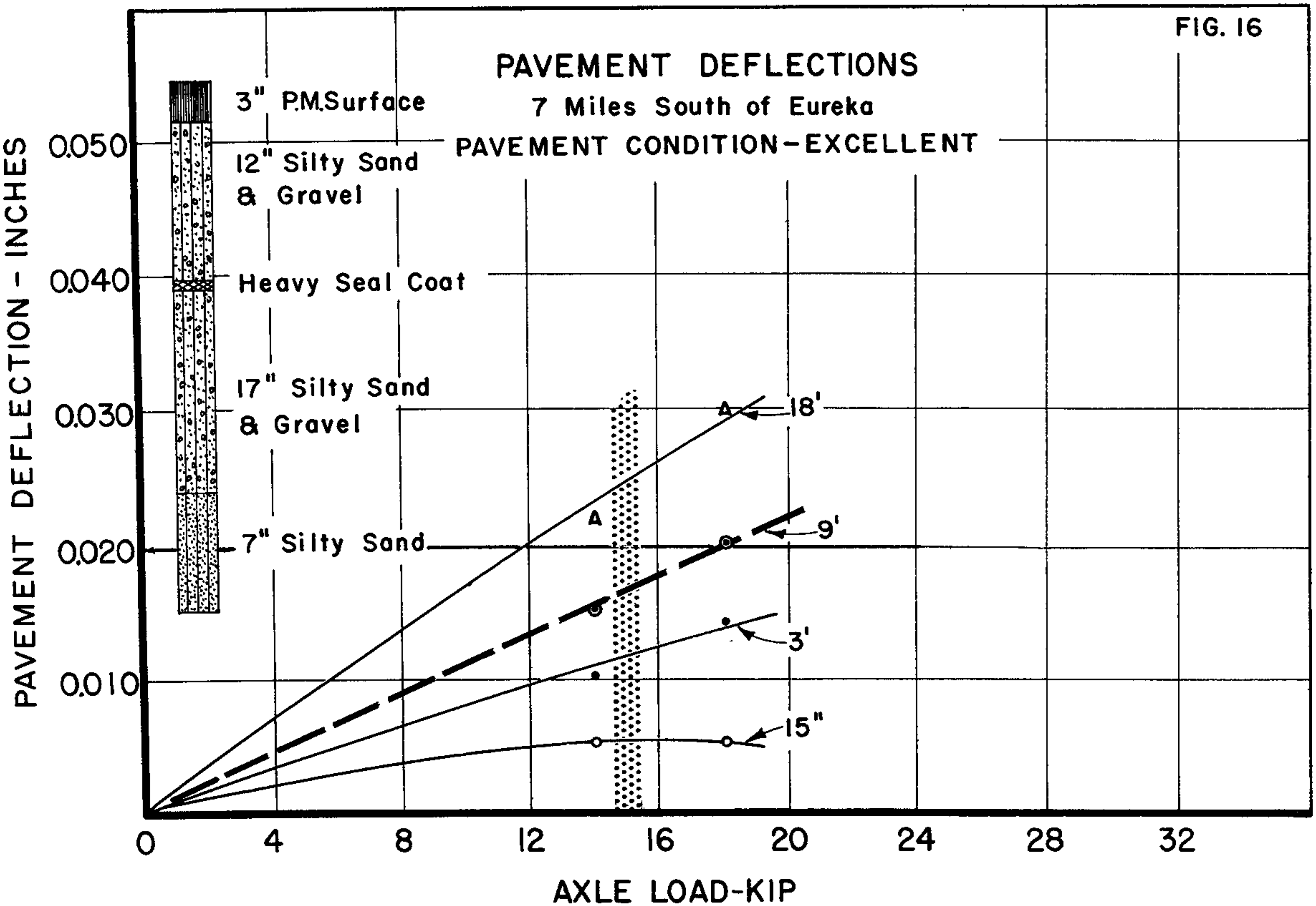

PAVEMENT DEFLECTIONS
5 Miles North of Eureka
CONCRETE PAVEMENT CONDITION-FAIR
7" PCC
7" Clayey
Sand & Gravel
Blue & Black
Bay Mud
18'
9'
15"
PAVEMENT DEFLECTION - INCHES
0.050
0.040
0.030
0.020
0.010
0
4
8
12
16
20
24
28
32
AXLE LOAD-KIP

FIG. 16


PAVEMENT DEFLECTIONS
7 Miles South of Eureka
PAVEMENT CONDITION-EXCELLENT
3" P.M.Surface
12" Silty Sand & Gravel
Heavy Seal Coat
17" Silty Sand & Gravel
7" Silty Sand
A
A
18'
9'
3'
15"
0.050
0.040
0.030
0.020
0.010
PAVEMENT DEFLECTION - INCHES
0
4
8
12
16
20
24
28
32
AXLE LOAD-KIP

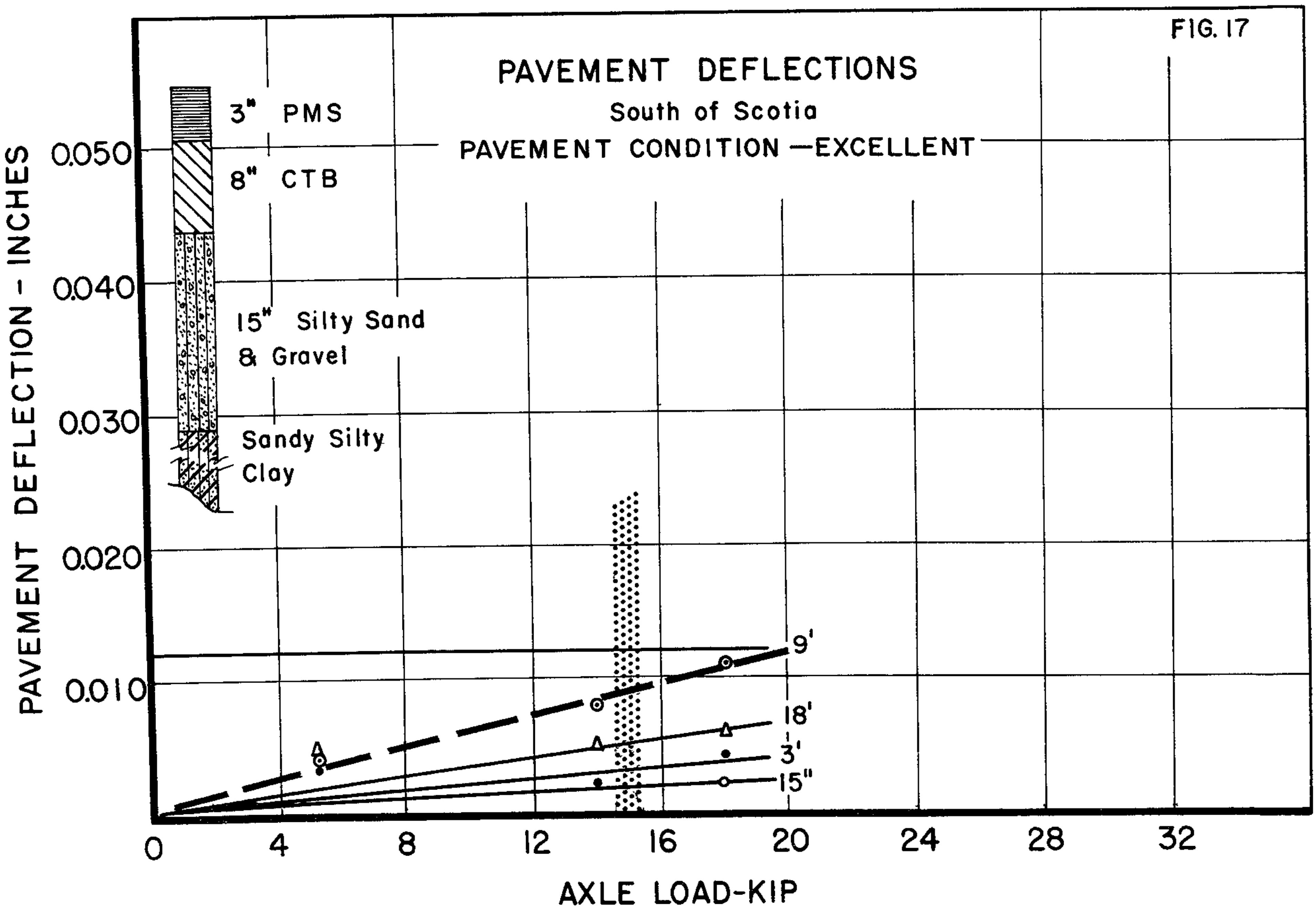


FIG. 17
PAVEMENT DEFLECTIONS
South of Scotia
PAVEMENT CONDITION —EXCELLENT
3" PMS
8" CTB
15" Silty Sand
& Gravel
Sandy Silty
Clay
9'
18'
3'
15"
PAVEMENT DEFLECTION - INCHES
0.050
0.040
0.030
0.020
0.010
0
4
8
12
16
20
24
28
32
AXLE LOAD-KIP

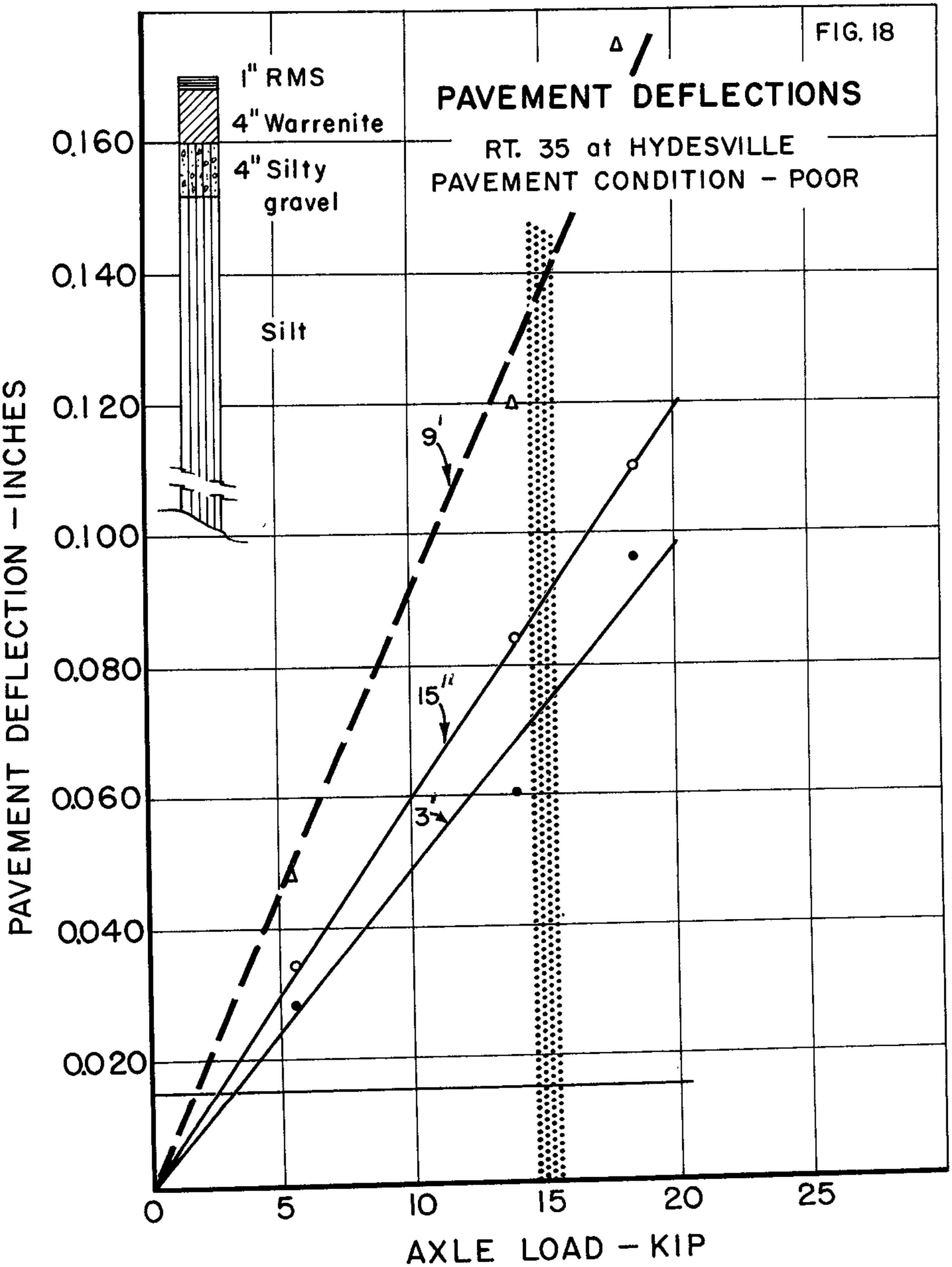


FIG. 18
PAVEMENT DEFLECTIONS
RT. 35 at HYDESVILLE
PAVEMENT CONDITION - POOR
1" RMS
4" Warrenite
4" Silty gravel
Silt
9'
15'
3'
0.160
0.140
0.120
0.100
0.080
0.060
0.040
0.020
0
5
10
15
20
25
PAVEMENT DEFLECTION - INCHES
AXLE LOAD - KIP

FIG. 19

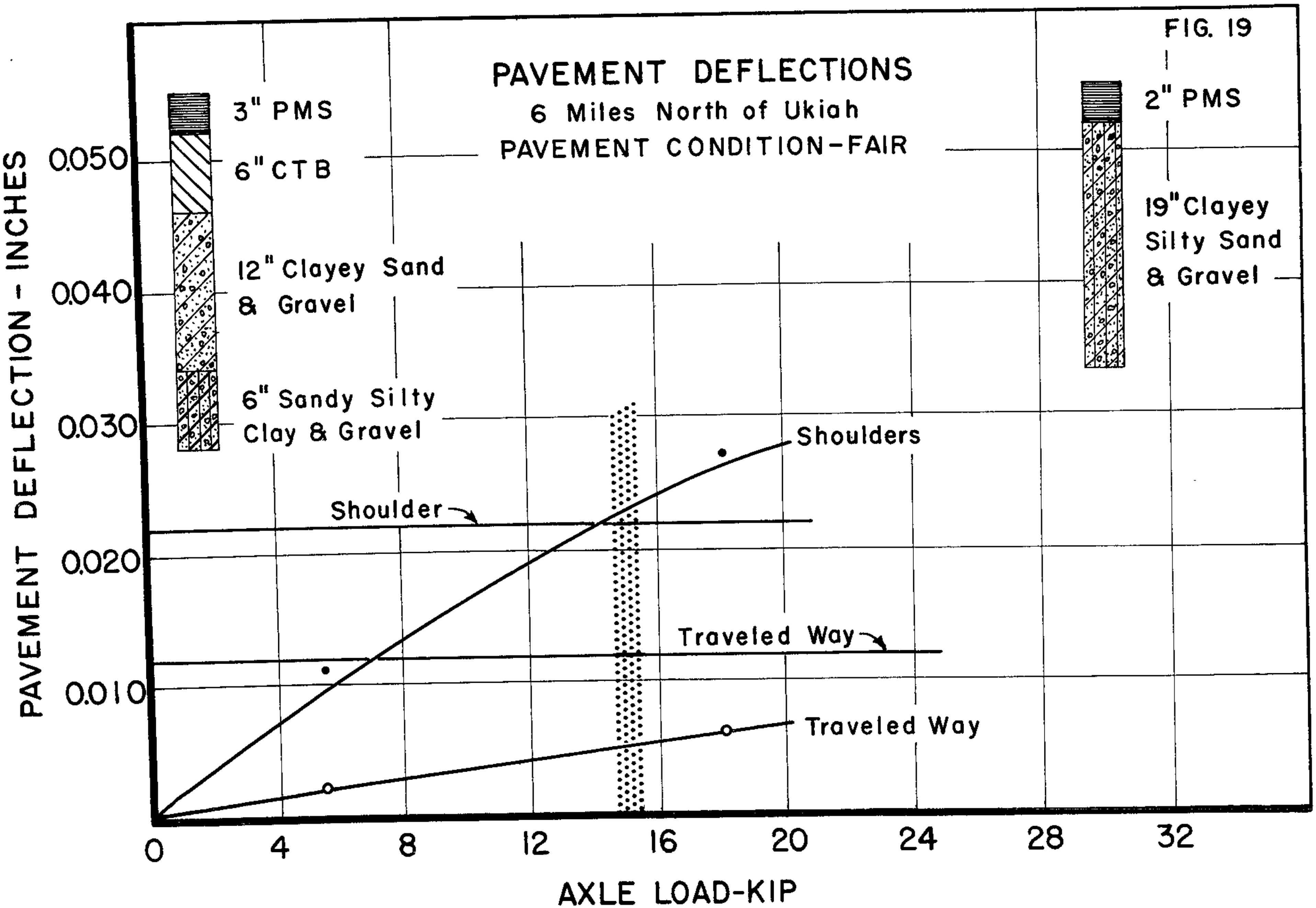
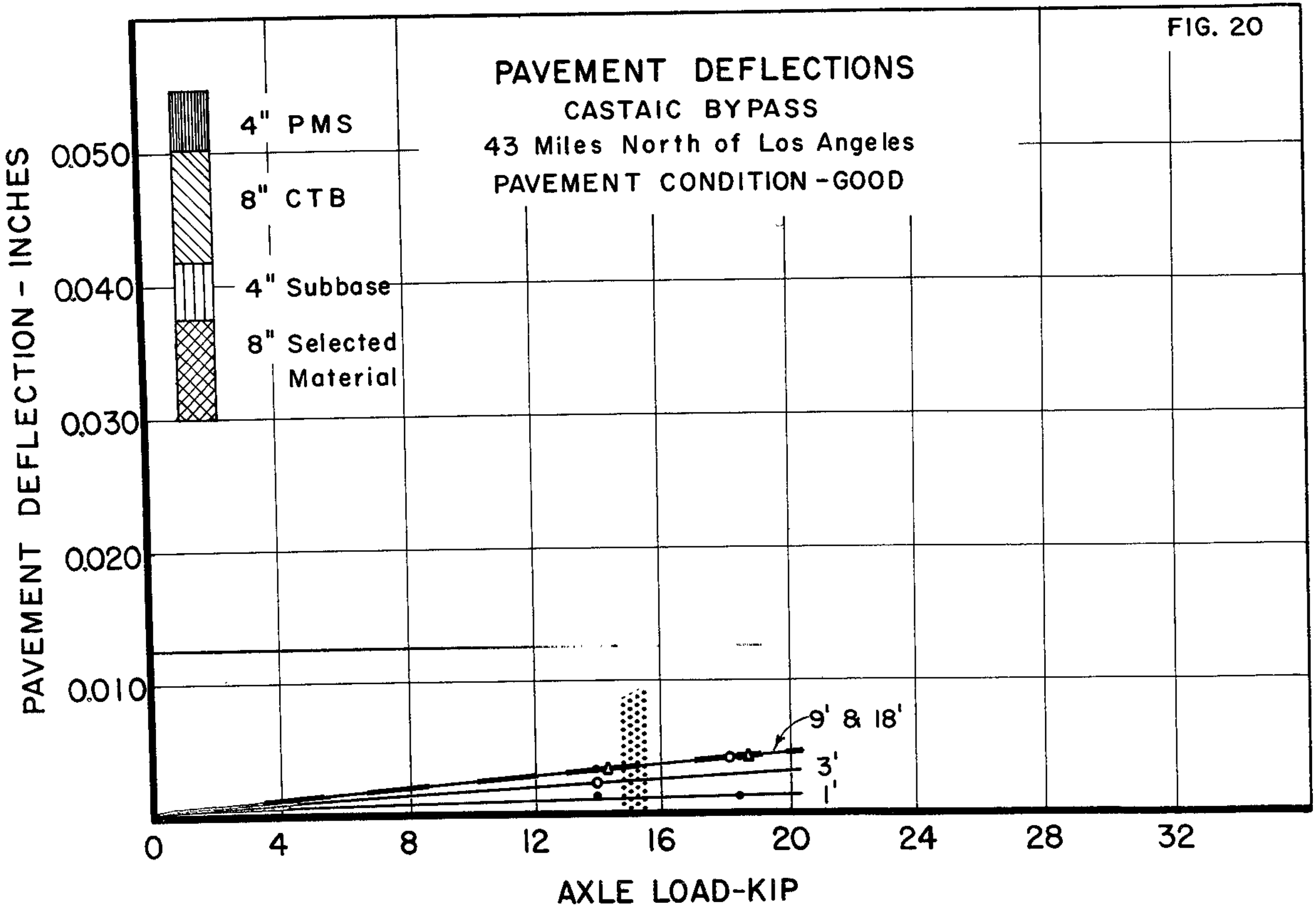
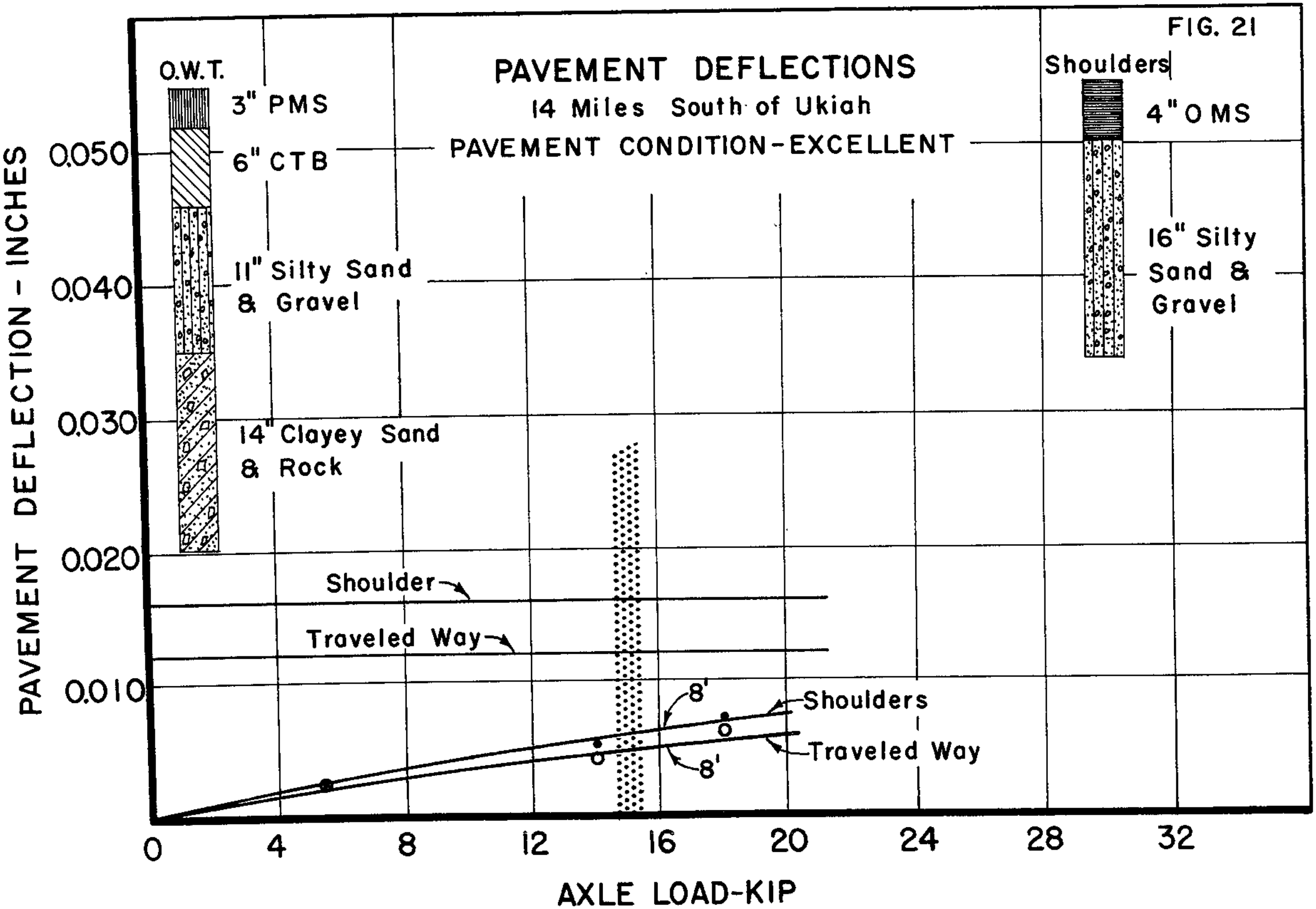

PAVEMENT DEFLECTIONS
6 Miles North of Ukiah
PAVEMENT CONDITION-FAIR
3" PMS
6" CTB
12" Clayey Sand & Gravel
6" Sandy Silty Clay & Gravel
2" PMS
19" Clayey Silty Sand & Gravel
Shoulders
Shoulder
Traveled Way
Traveled Way
PAVEMENT DEFLECTION - INCHES
0.050
0.040
0.030
0.020
0.010
0
4
8
12
16
20
24
28
32
AXLE LOAD-KIP

FIG. 20



PAVEMENT DEFLECTIONS
CASTAIC BYPASS
43 Miles North of Los Angeles
PAVEMENT CONDITION - GOOD
4" PMS
8" CTB
4" Subbase
8" Selected Material
9' & 18'
3'
1'
PAVEMENT DEFLECTION - INCHES
0.050
0.040
0.030
0.020
0.010
0
4
8
12
16
20
24
28
32
AXLE LOAD-KIP

FIG. 21


PAVEMENT DEFLECTIONS
14 Miles South of Ukiah
PAVEMENT CONDITION - EXCELLENT
O.W.T.
3" PMS
6" CTB
11" Silty Sand & Gravel
14" Clayey Sand & Rock
Shoulders
4" OMS
16" Silty Sand & Gravel
Shoulder
Traveled Way
Shoulders
Traveled Way
8'
8'
0.050
0.040
0.030
0.020
0.010
PAVEMENT DEFLECTION - INCHES
0
4
8
12
16
20
24
28
32
AXLE LOAD-KIP

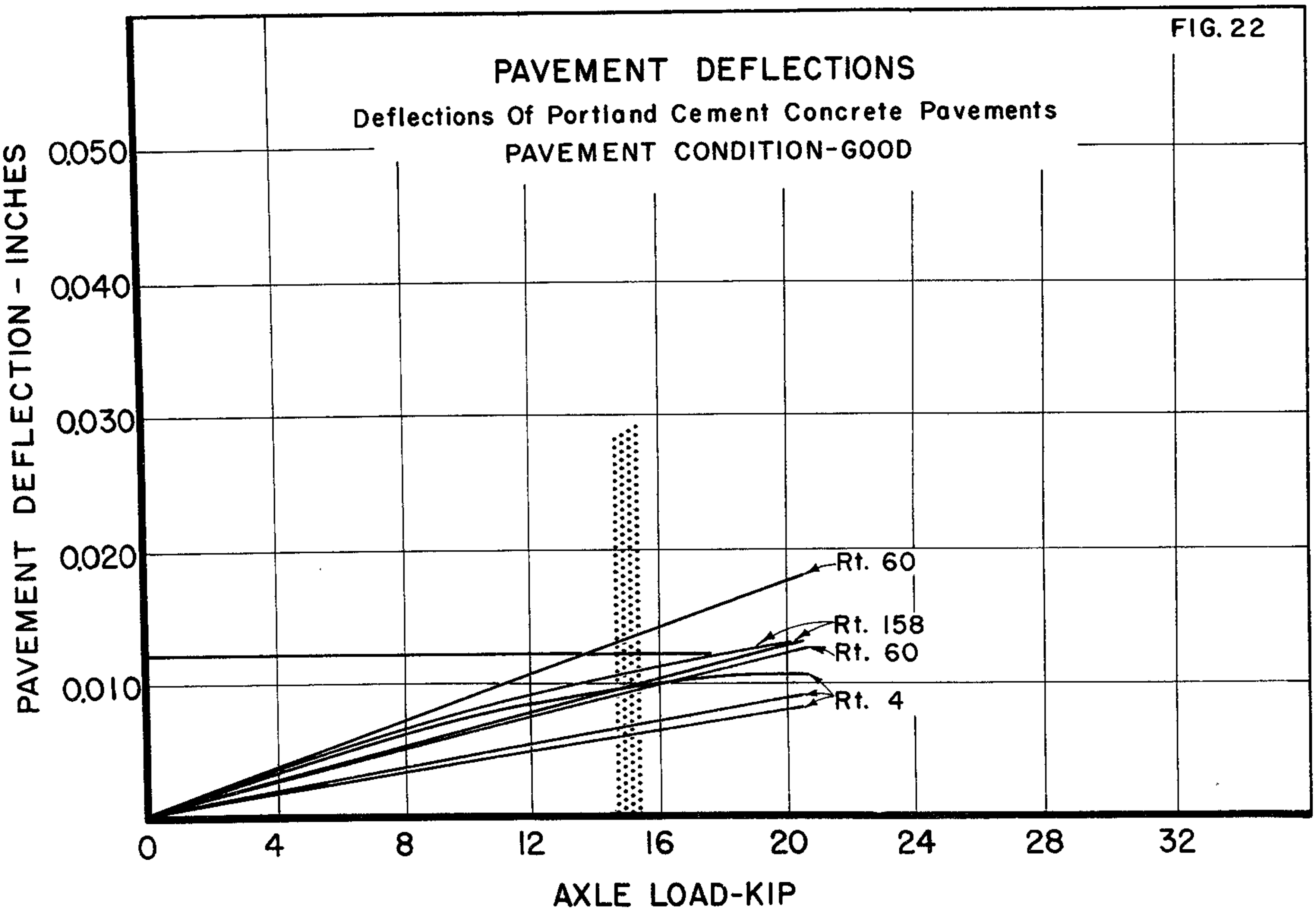
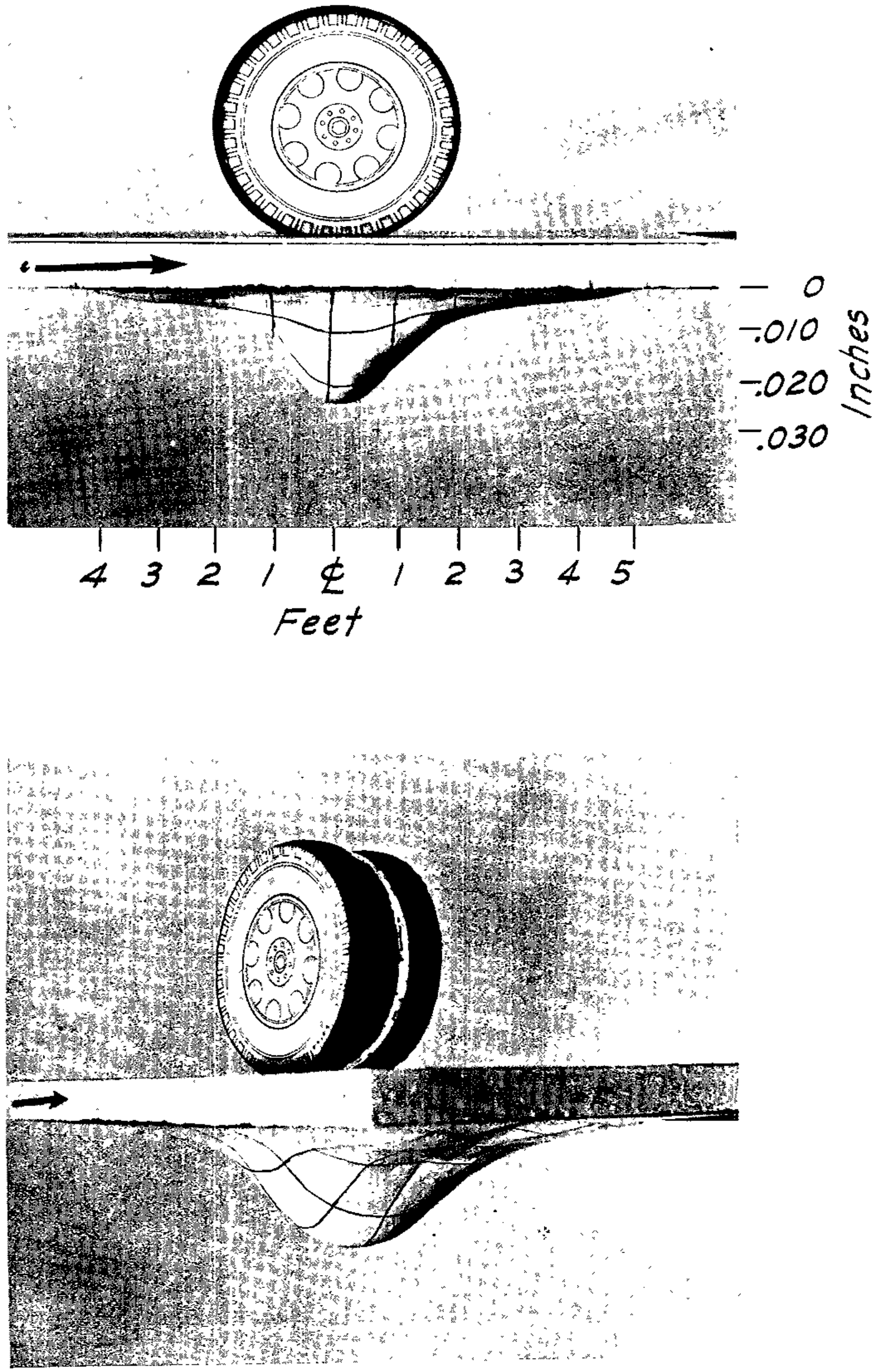


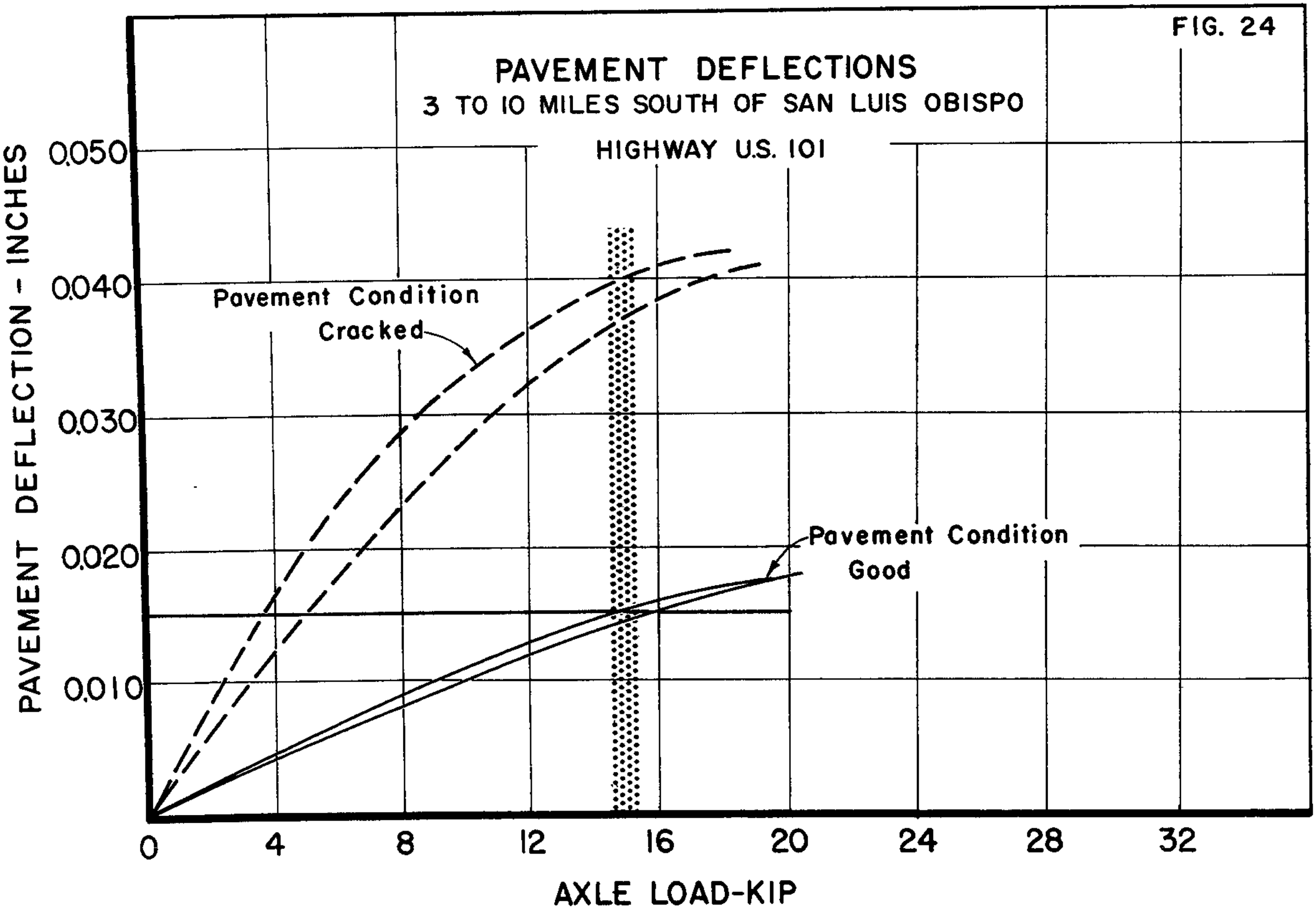
FIG. 22
PAVEMENT DEFLECTIONS
Deflections Of Portland Cement Concrete Pavements
PAVEMENT CONDITION-GOOD
PAVEMENT DEFLECTION - INCHES
0.050
0.040
0.030
0.020
0.010
0
4
8
12
16
20
24
28
32
AXLE LOAD-KIP
Rt. 60
Rt. 158
Rt. 60
Rt. 4



Model Showing Deflection
Pattern Under a Dual Wheel Load

Fig. 23

FIG. 24



PAVEMENT DEFLECTIONS
3 TO 10 MILES SOUTH OF SAN LUIS OBISPO
HIGHWAY U.S. 101
Pavement Condition
Cracked
Pavement Condition
Good
PAVEMENT DEFLECTION - INCHES
0.050
0.040
0.030
0.020
0.010
0
4
8
12
16
20
24
28
32
AXLE LOAD-KIP

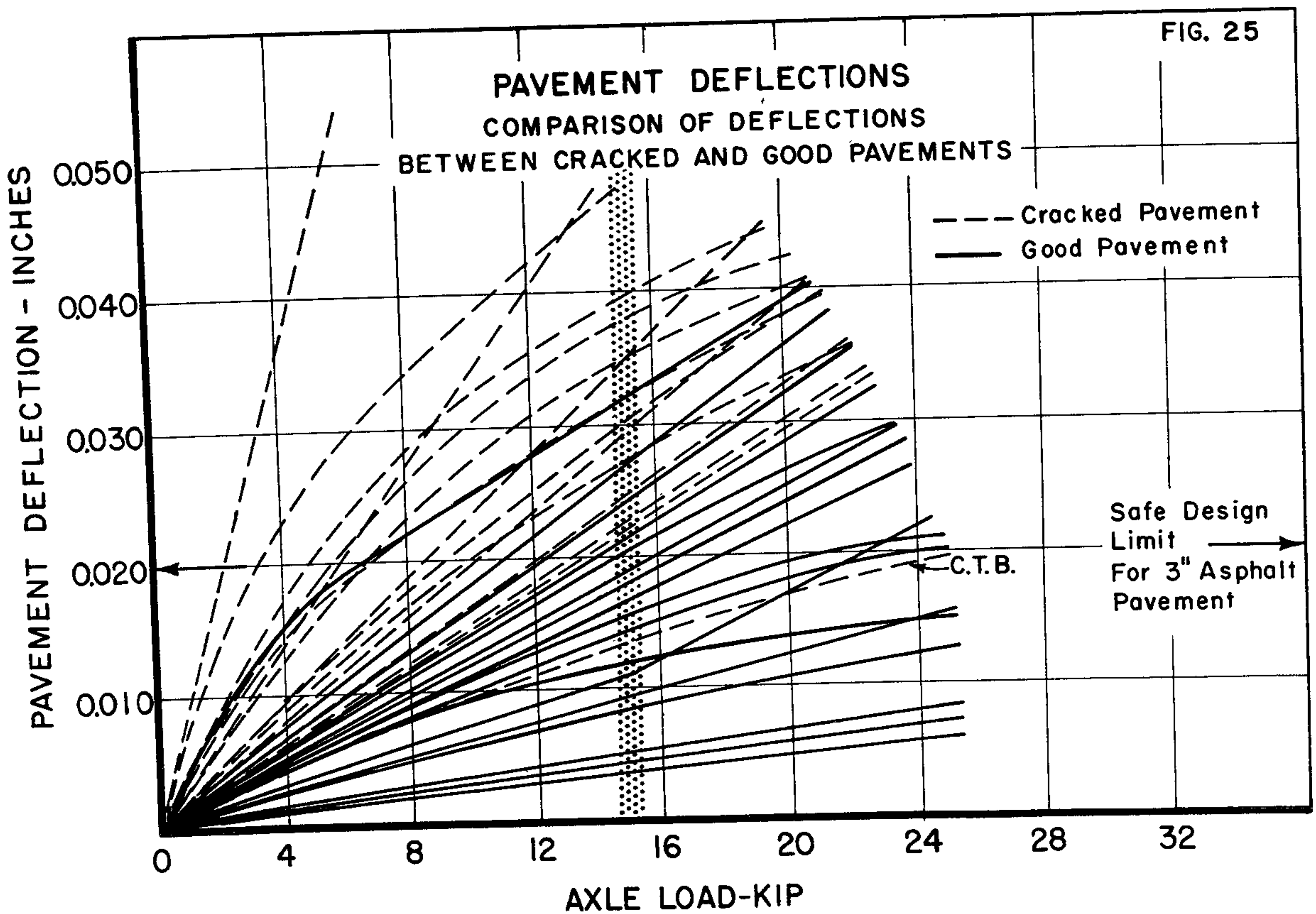


FIG. 25
PAVEMENT DEFLECTIONS
COMPARISON OF DEFLECTIONS
BETWEEN CRACKED AND GOOD PAVEMENTS
Cracked Pavement
Good Pavement
C.T.B.
Safe Design
Limit
For 3" Asphalt
Pavement
PAVEMENT DEFLECTION - INCHES
0.050
0.040
0.030
0.020
0.010
0
4
8
12
16
20
24
28
32
AXLE LOAD-KIP

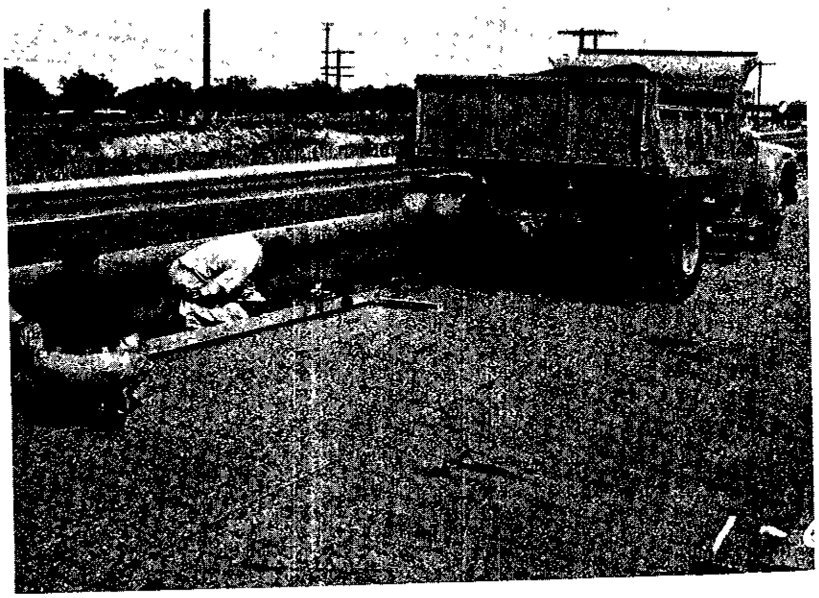
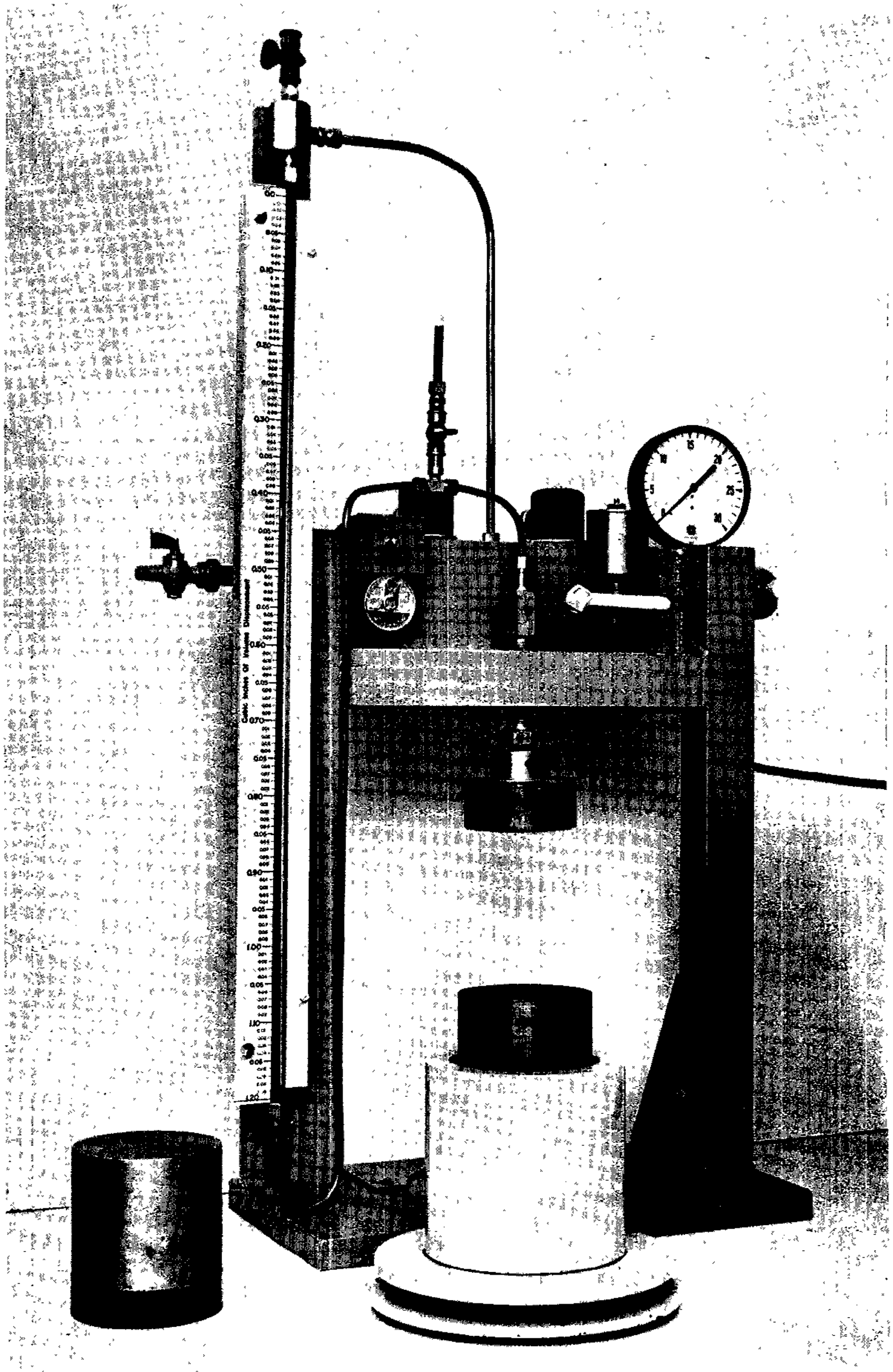


Figure 26

Benkelman Beam being used.



Resiliometer

Figure 27

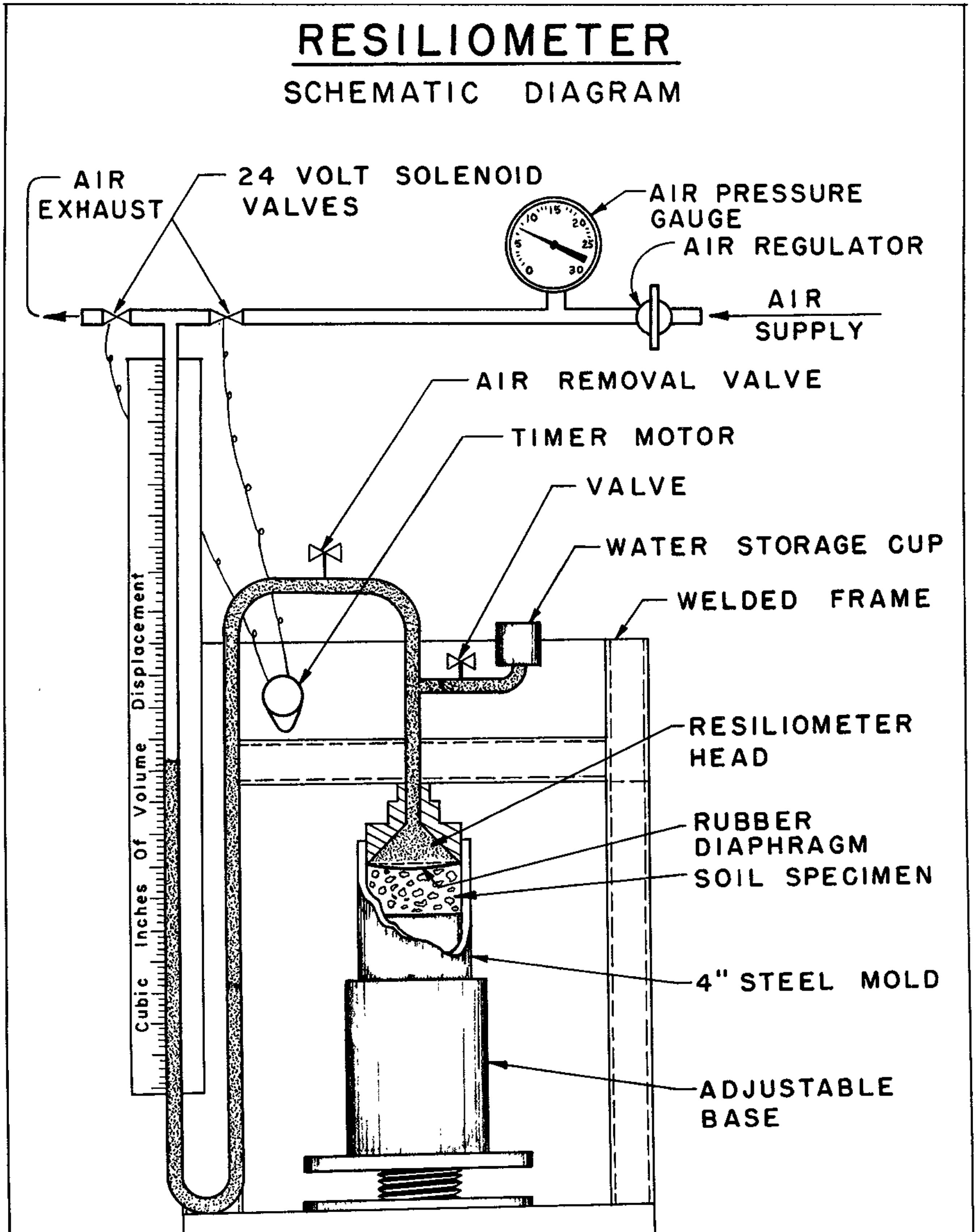
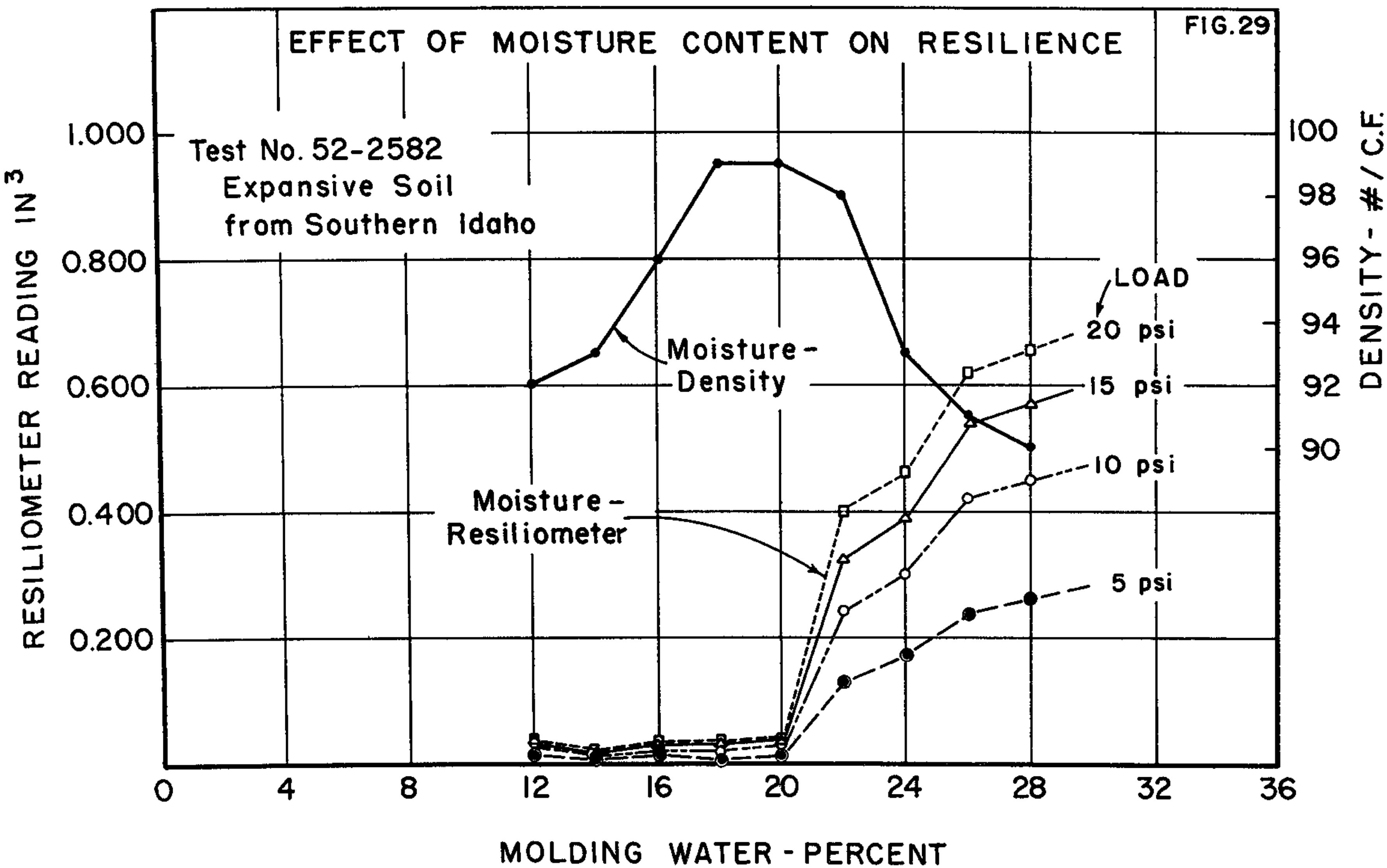

RESILIOMETER
SCHEMATIC DIAGRAM
AIR EXHAUST
24 VOLT SOLENOID VALVES
AIR PRESSURE GAUGE
AIR REGULATOR
AIR SUPPLY
AIR REMOVAL VALVE
TIMER MOTOR
VALVE
WATER STORAGE CUP
WELDED FRAME
RESILIOMETER HEAD
RUBBER DIAPHRAGM
SOIL SPECIMEN
4" STEEL MOLD
ADJUSTABLE BASE
Cubic Inches Of Volume Displacement

Fig. 28



EFFECT OF MOISTURE CONTENT ON RESILIENCE
FIG.29
Test No. 52-2582
Expansive Soil
from Southern Idaho
Moisture - Density
Moisture - Resiliometer
LOAD
20 psi
15 psi
10 psi
5 psi
RESILIOMETER READING IN^3
1.000
0.800
0.600
0.400
0.200
DENSITY - #/C.F.
100
98
96
94
92
90
0
4
8
12
16
20
24
28
32
36
MOLDING WATER - PERCENT

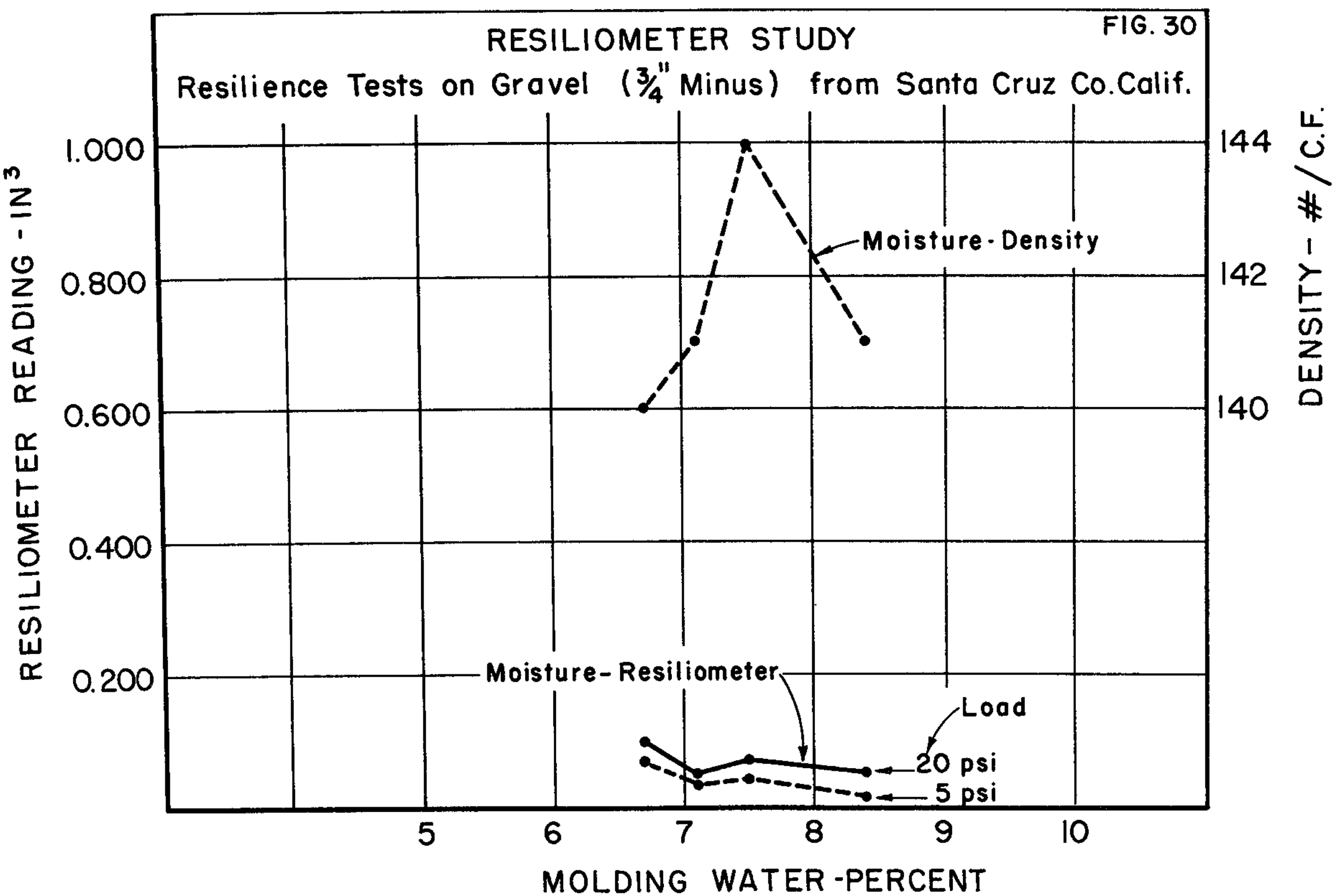


FIG. 30
RESILIOMETER STUDY
Resilience Tests on Gravel (3/4" Minus) from Santa Cruz Co. Calif.
Moisture - Density
Moisture - Resiliometer
Load
20 psi
5 psi
1.000
0.800
0.600
0.400
0.200
RESILIOMETER READING - IN³
144
142
140
DENSITY - #/C.F.
5
6
7
8
9
10
MOLDING WATER - PERCENT

BITUMINOUS PAVEMENT FATIGUE TESTER

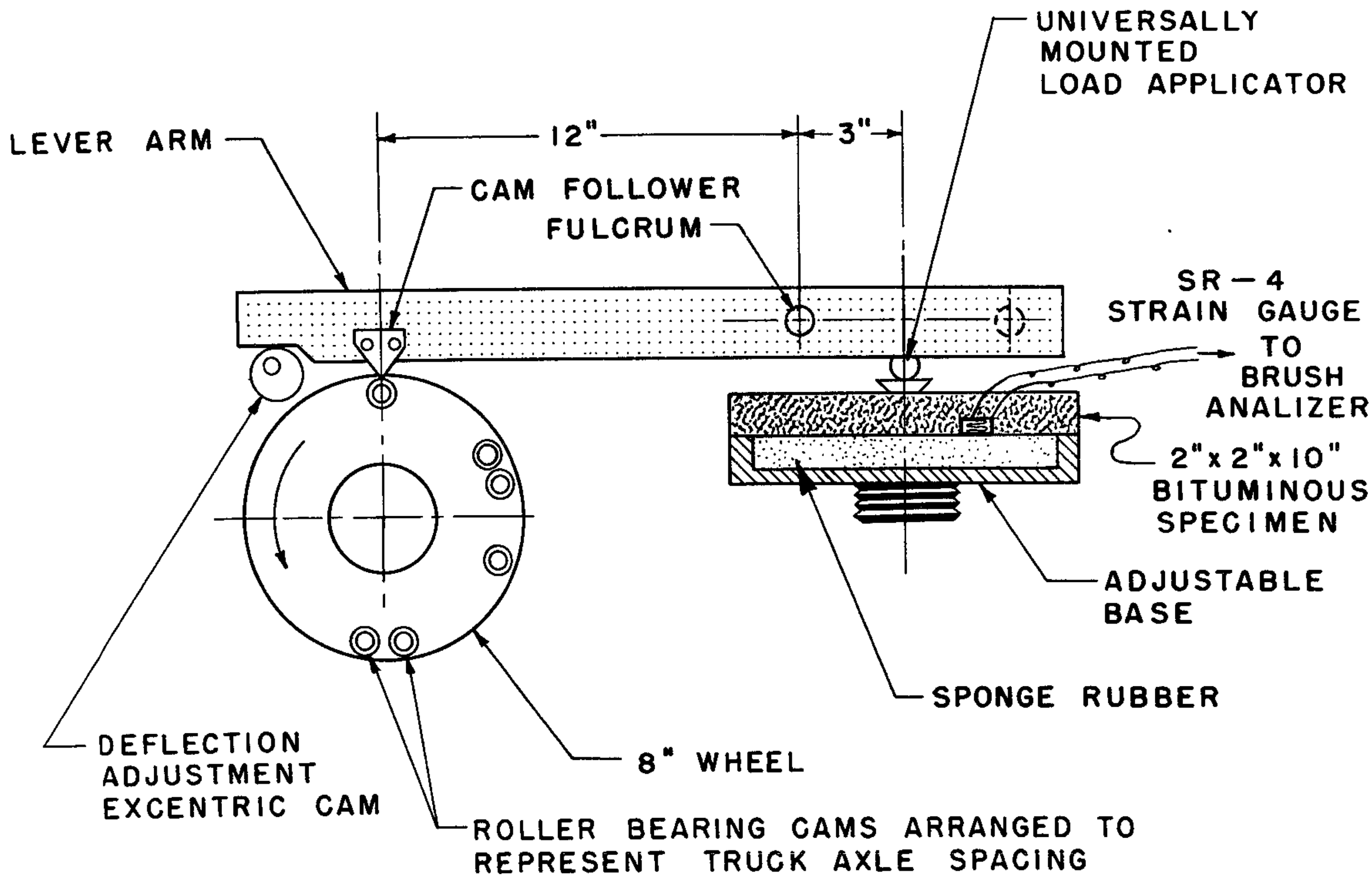
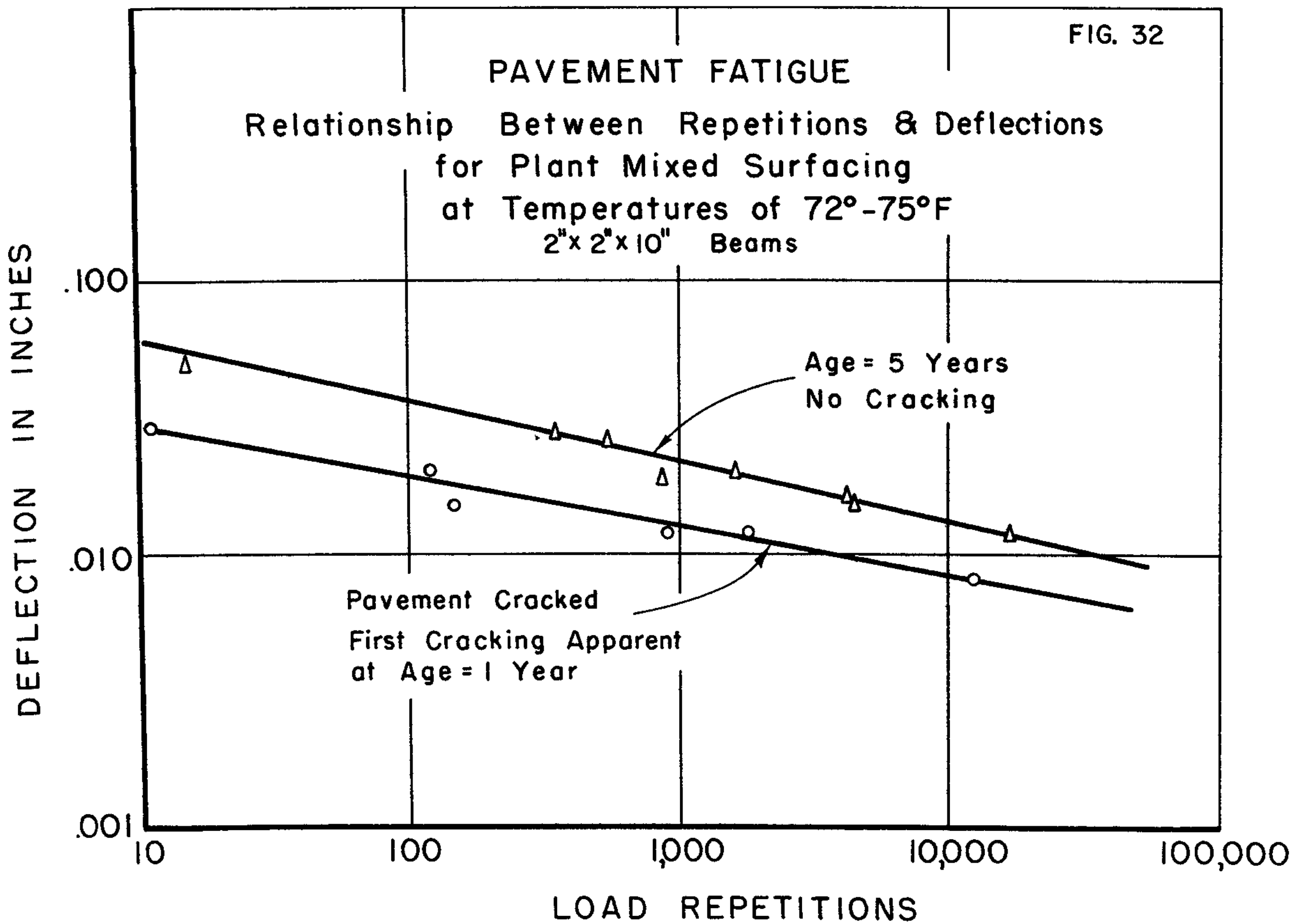


Fig. 31

FIG. 32



PAVEMENT FATIGUE
Relationship Between Repetitions & Deflections
for Plant Mixed Surfacing
at Temperatures of 72°-75°F
2"x 2"x 10" Beams
Age = 5 Years
No Cracking
Pavement Cracked
First Cracking Apparent
at Age = 1 Year
.100
.010
.001
10
100
1,000
10,000
100,000
LOAD REPETITIONS
DEFLECTION IN INCHES

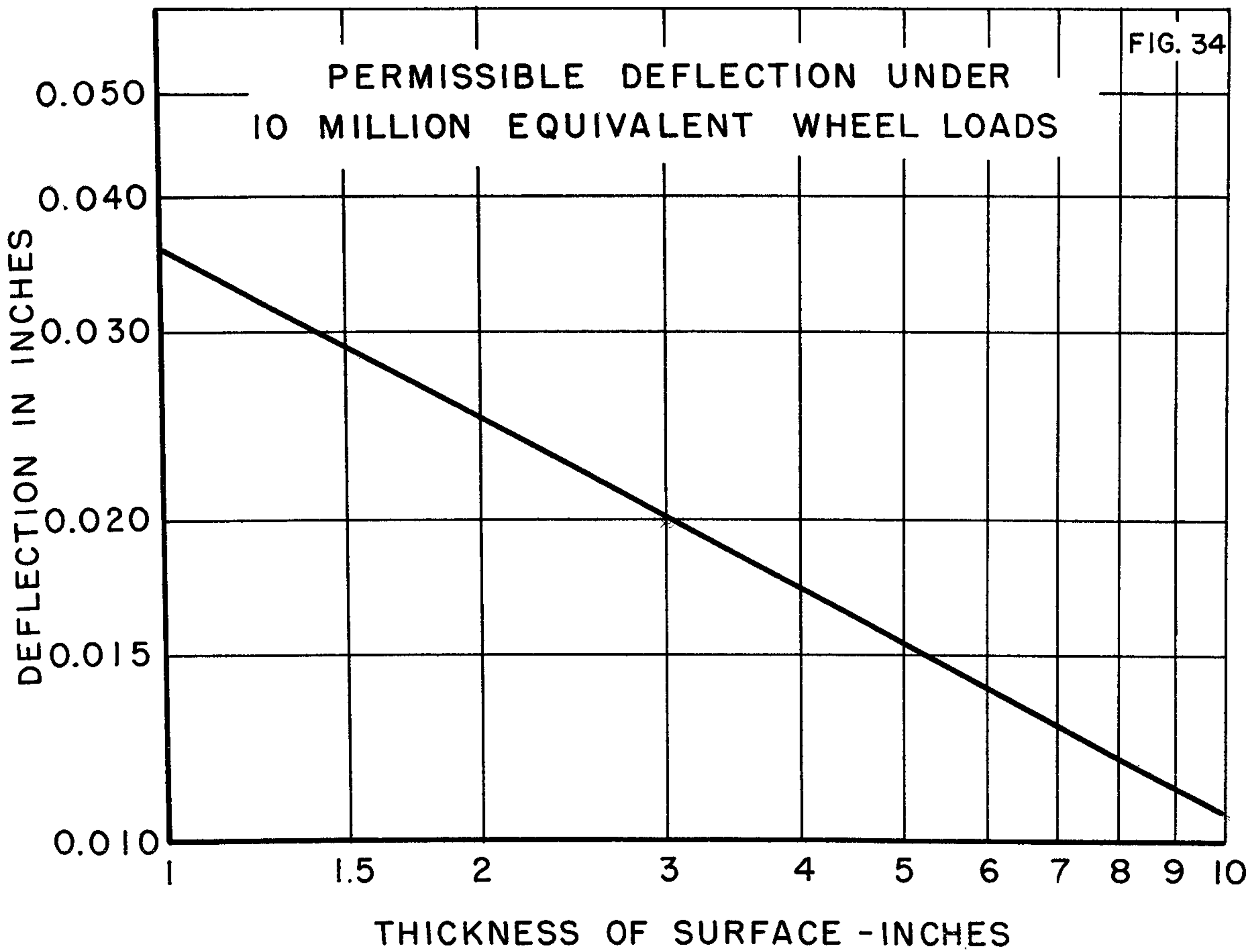


FIG. 34
PERMISSIBLE DEFLECTION UNDER
10 MILLION EQUIVALENT WHEEL LOADS
DEFLECTION IN INCHES
0.050
0.040
0.030
0.020
0.015
0.010
1
1.5
2
3
4
5
6
7
8
9
10
THICKNESS OF SURFACE - INCHES

FIG. 35

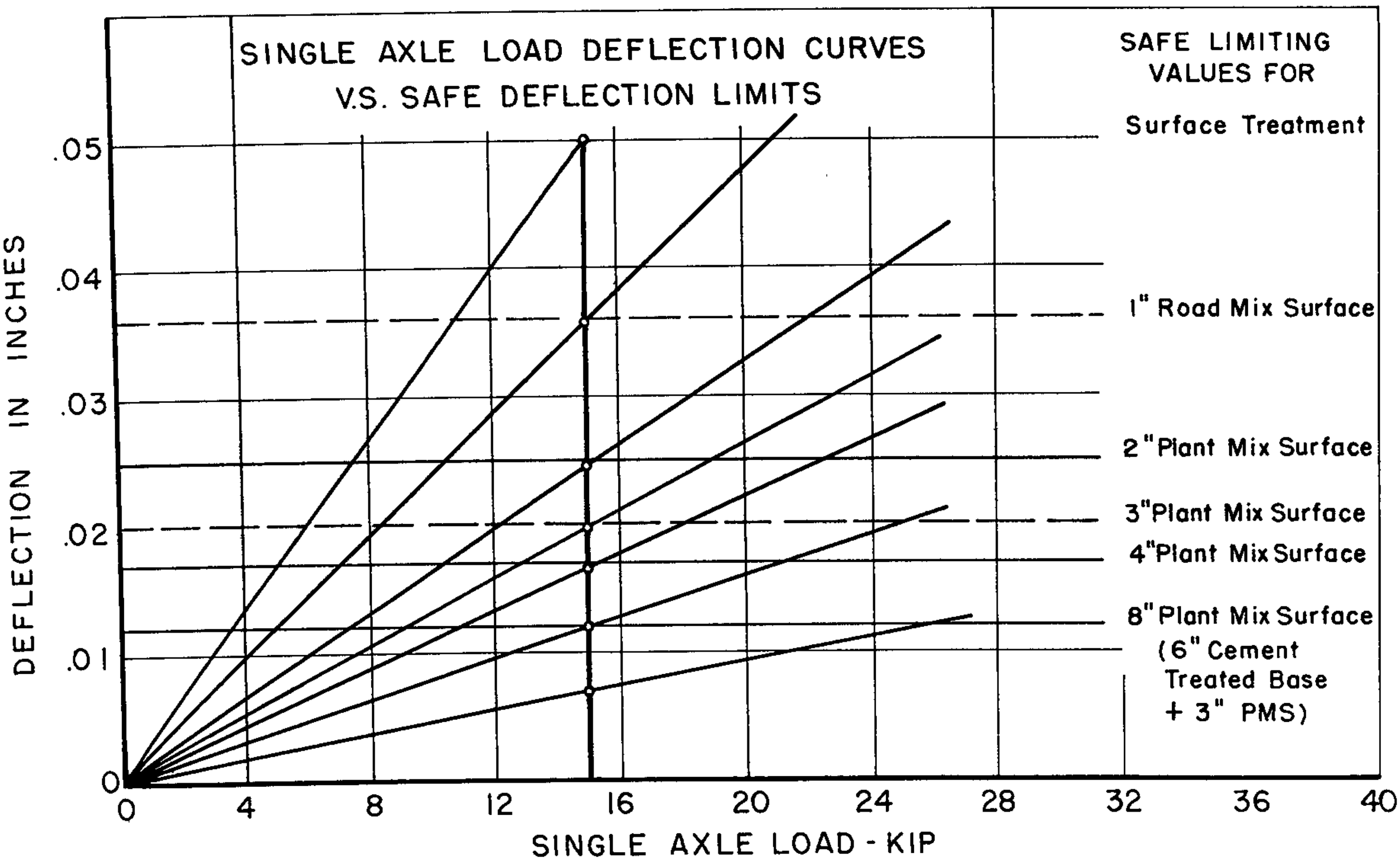
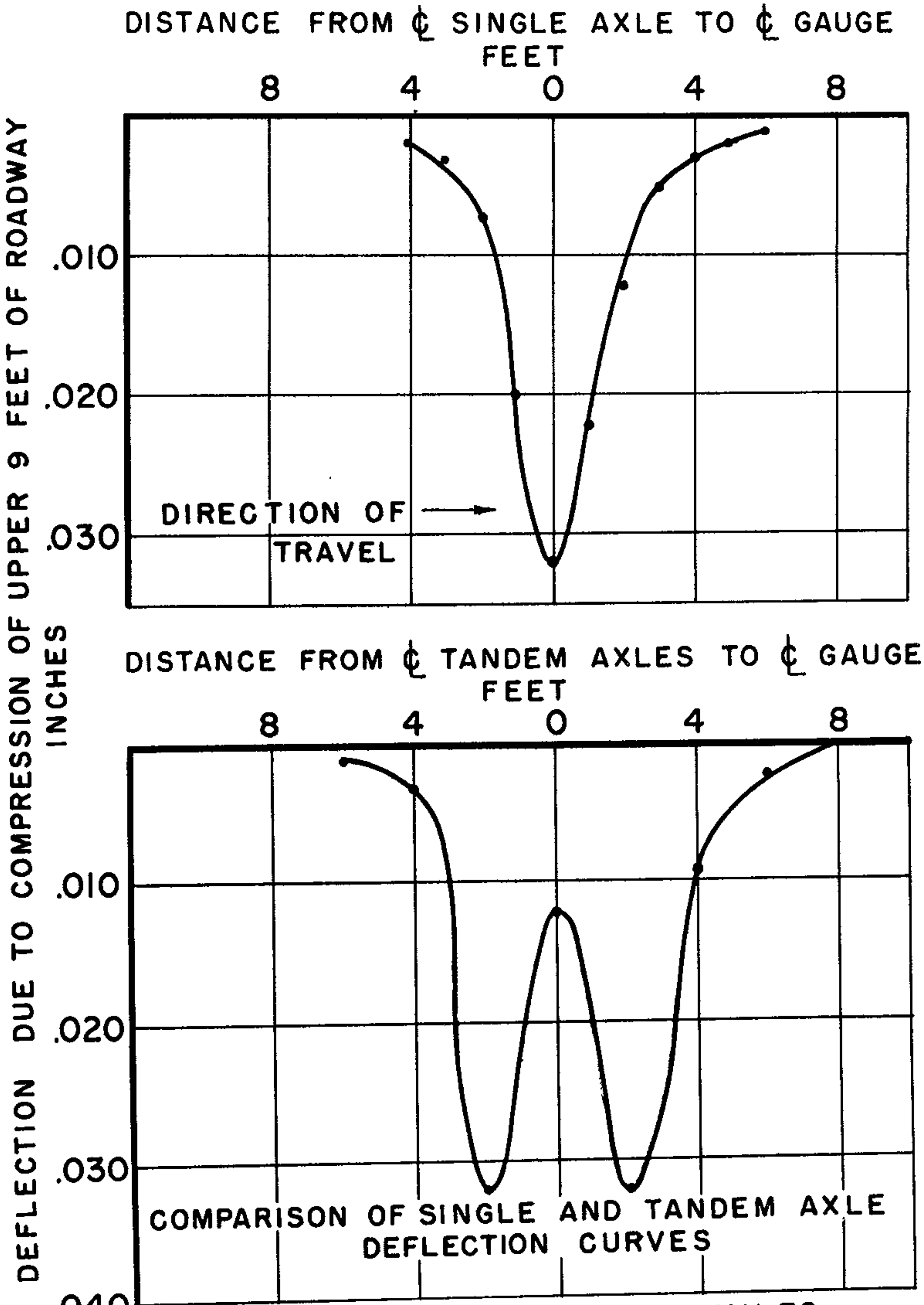
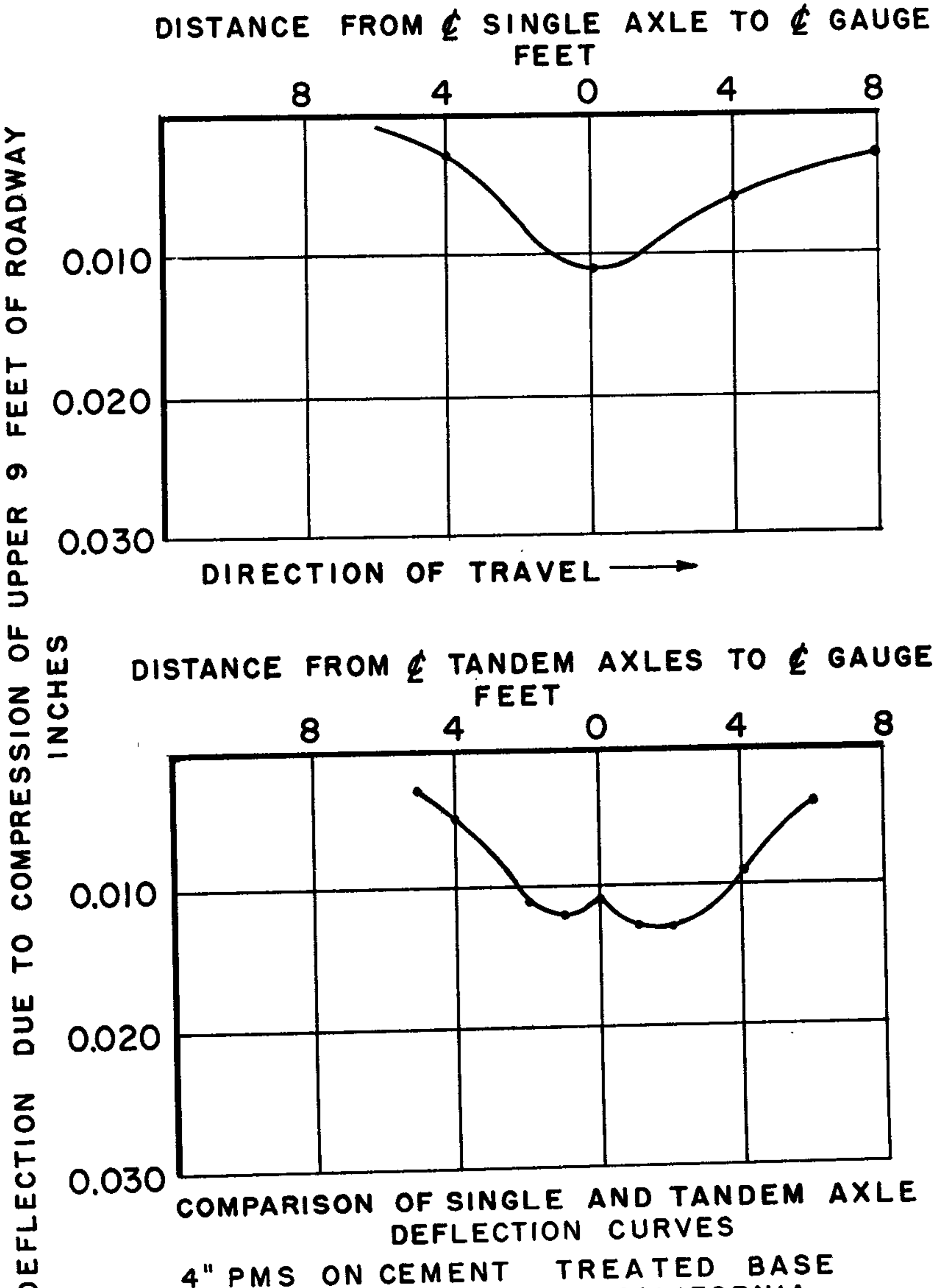

SINGLE AXLE LOAD DEFLECTION CURVES
V.S. SAFE DEFLECTION LIMITS
SAFE LIMITING VALUES FOR
Surface Treatment
1" Road Mix Surface
2" Plant Mix Surface
3" Plant Mix Surface
4" Plant Mix Surface
8" Plant Mix Surface
(6" Cement Treated Base + 3" PMS)
DEFLECTION IN INCHES
.05
.04
.03
.02
.01
0
0
4
8
12
16
20
24
28
32
36
40
SINGLE AXLE LOAD - KIP

Fig. 36



4" PMS ON GRAVEL BASE, 63 MILES NORTH OF LOS ANGELES 9' REFERENCE RODS

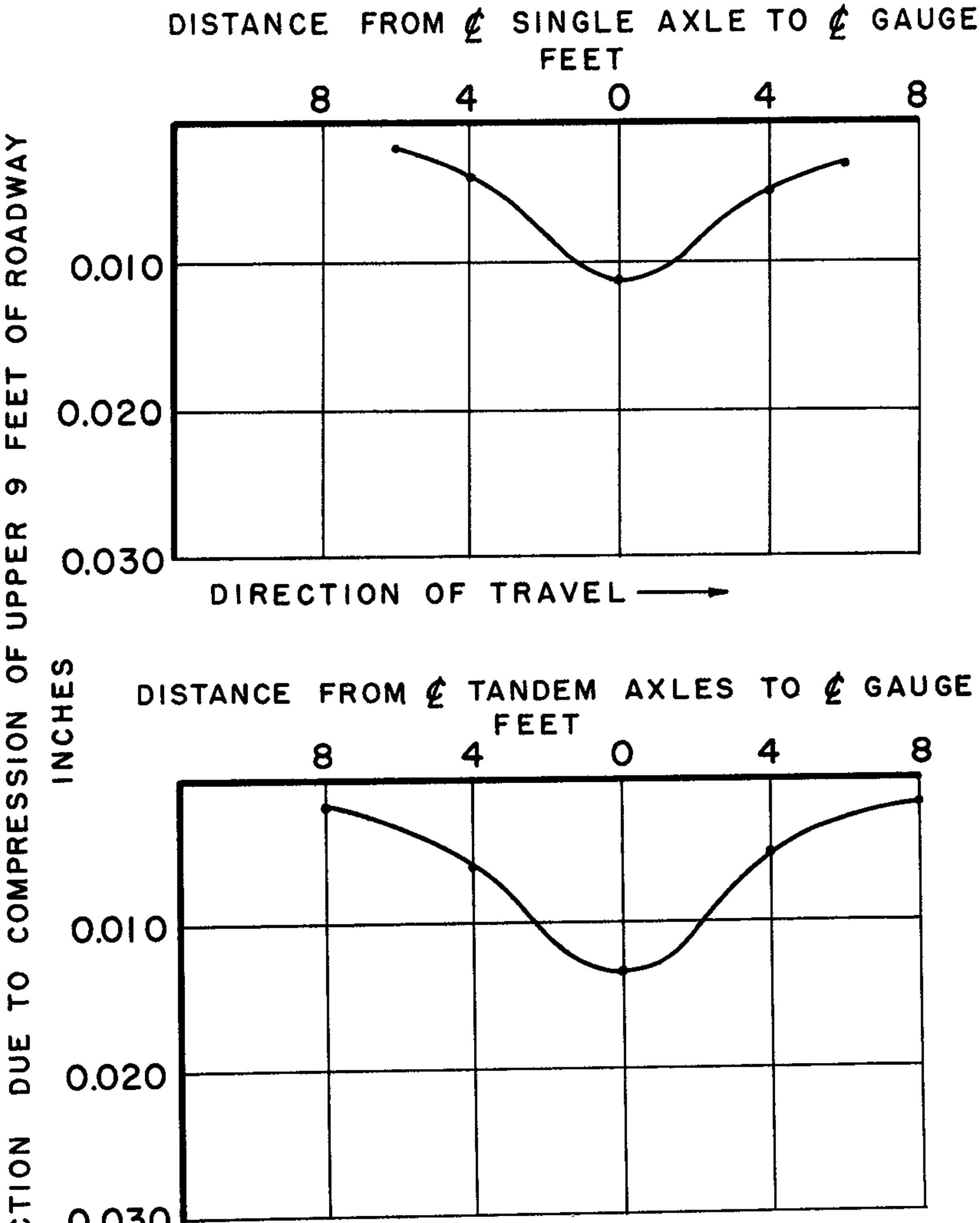
FIG. 37



COMPARISON OF SINGLE AND TANDEM AXLE DEFLECTION CURVES

4" PMS ON CEMENT TREATED BASE
SOUTH OF SCOTIA, CALIFORNIA

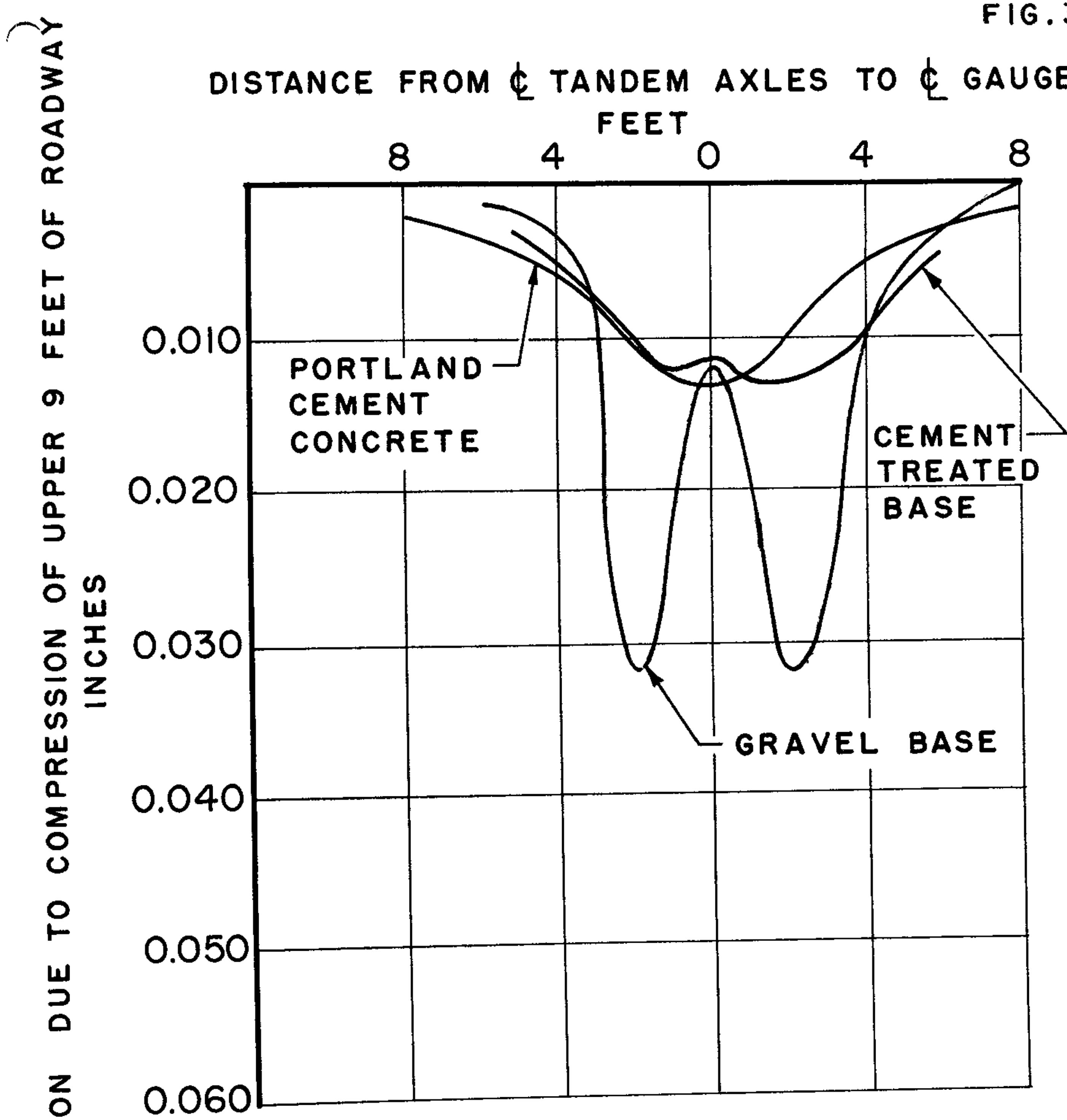
FIG. 38



COMPARISON OF SINGLE AND TANDEM AXLE DEFLECTION CURVES

8" PORTLAND CEMENT CONCRETE PAVEMENT ON SEPULVEDA BLVD. IN LOS ANGELES CALIF.

FIG. 39



COMPARISON OF TANDEM AXLE DEFLECTION CURVES ON VARIOUS TYPES OF PAVEMENTS

FIG. 40

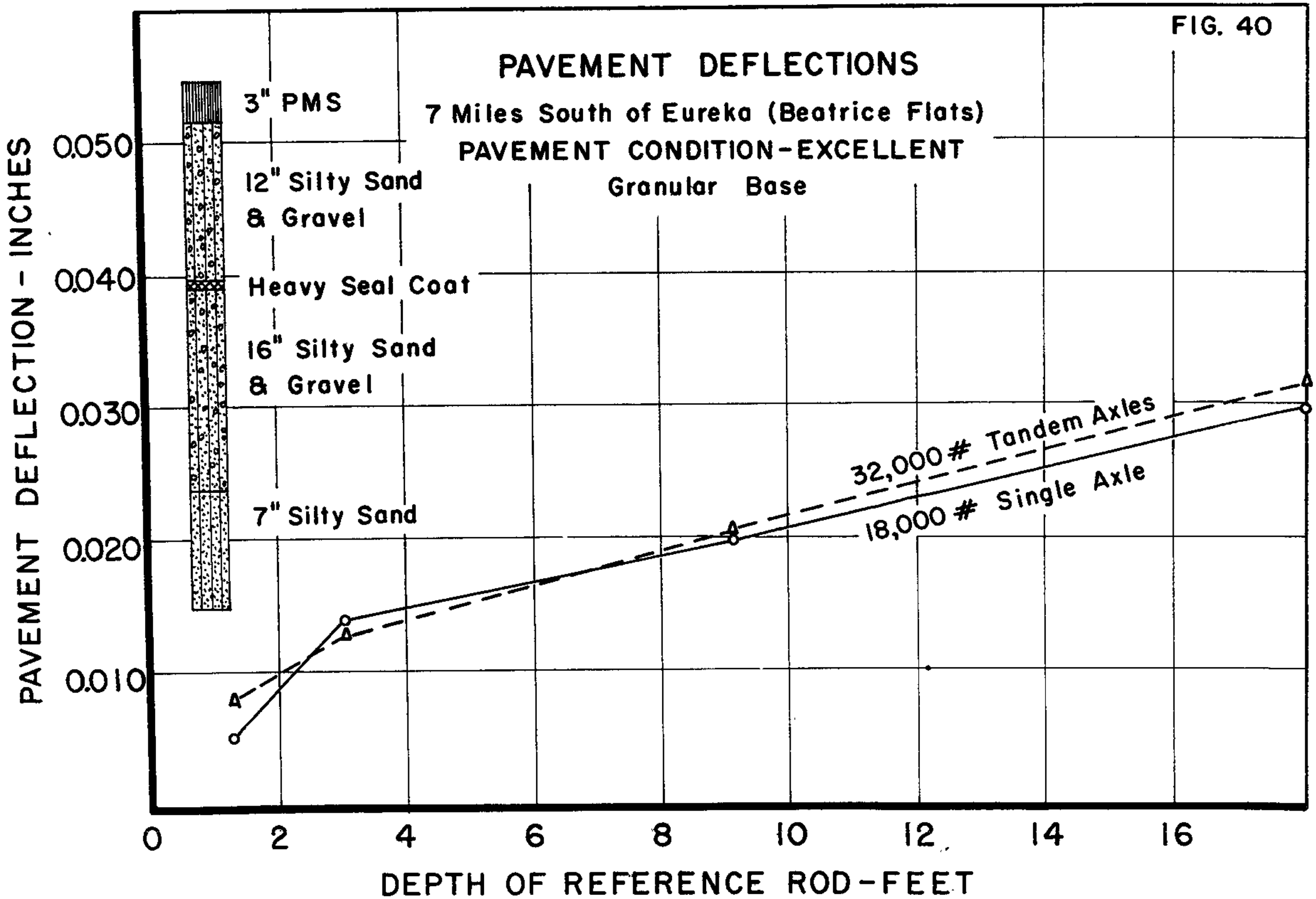
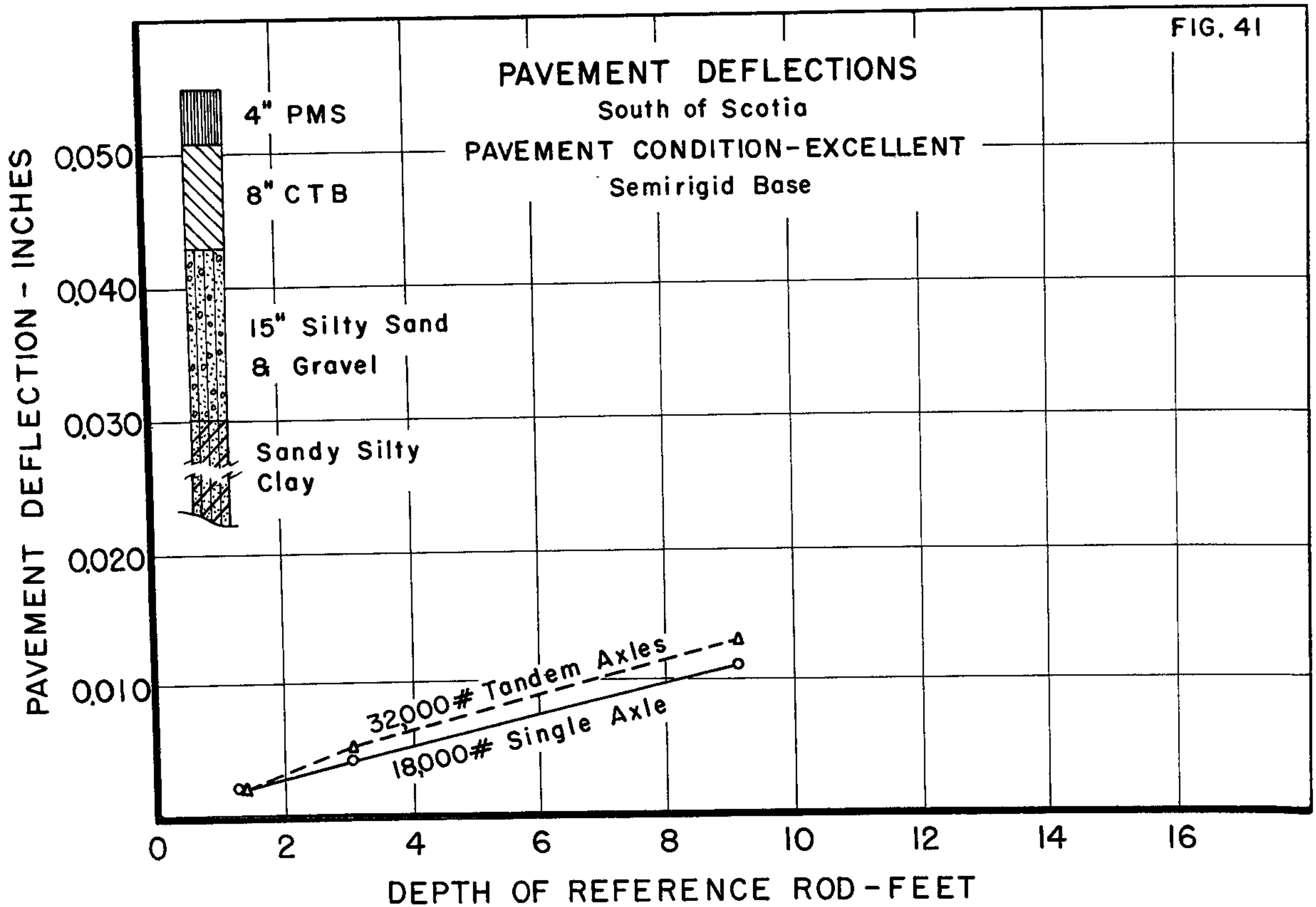
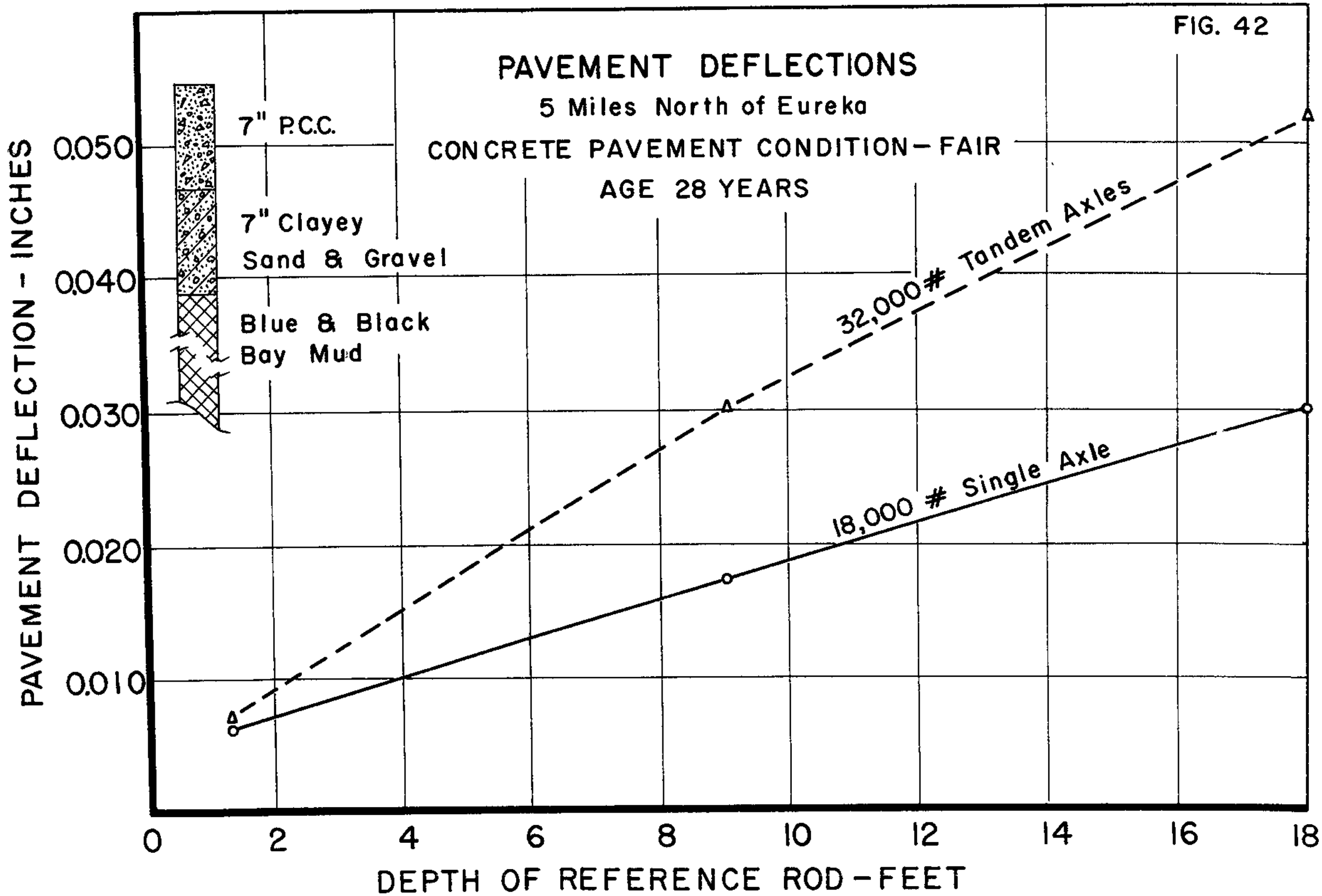

PAVEMENT DEFLECTIONS
7 Miles South of Eureka (Beatrice Flats)
PAVEMENT CONDITION-EXCELLENT
Granular Base
3" PMS
12" Silty Sand & Gravel
Heavy Seal Coat
16" Silty Sand & Gravel
7" Silty Sand
32,000 # Tandem Axles
18,000 # Single Axle
0.050
0.040
0.030
0.020
0.010
0
2
4
6
8
10
12
14
16
PAVEMENT DEFLECTION - INCHES
DEPTH OF REFERENCE ROD-FEET

FIG. 41



PAVEMENT DEFLECTIONS
South of Scotia
PAVEMENT CONDITION-EXCELLENT
Semirigid Base
4" PMS
8" CTB
15" Silty Sand
& Gravel
Sandy Silty
Clay
32,000# Tandem Axles
18,000# Single Axle
PAVEMENT DEFLECTION - INCHES
0.050
0.040
0.030
0.020
0.010
0
2
4
6
8
10
12
14
16
DEPTH OF REFERENCE ROD-FEET

FIG. 42


PAVEMENT DEFLECTIONS
5 Miles North of Eureka
CONCRETE PAVEMENT CONDITION – FAIR
AGE 28 YEARS
7" P.C.C.
7" Clayey Sand & Gravel
Blue & Black Bay Mud
32,000 # Tandem Axles
18,000 # Single Axle
PAVEMENT DEFLECTION - INCHES
0.050
0.040
0.030
0.020
0.010
0
2
4
6
8
10
12
14
16
18
DEPTH OF REFERENCE ROD – FEET

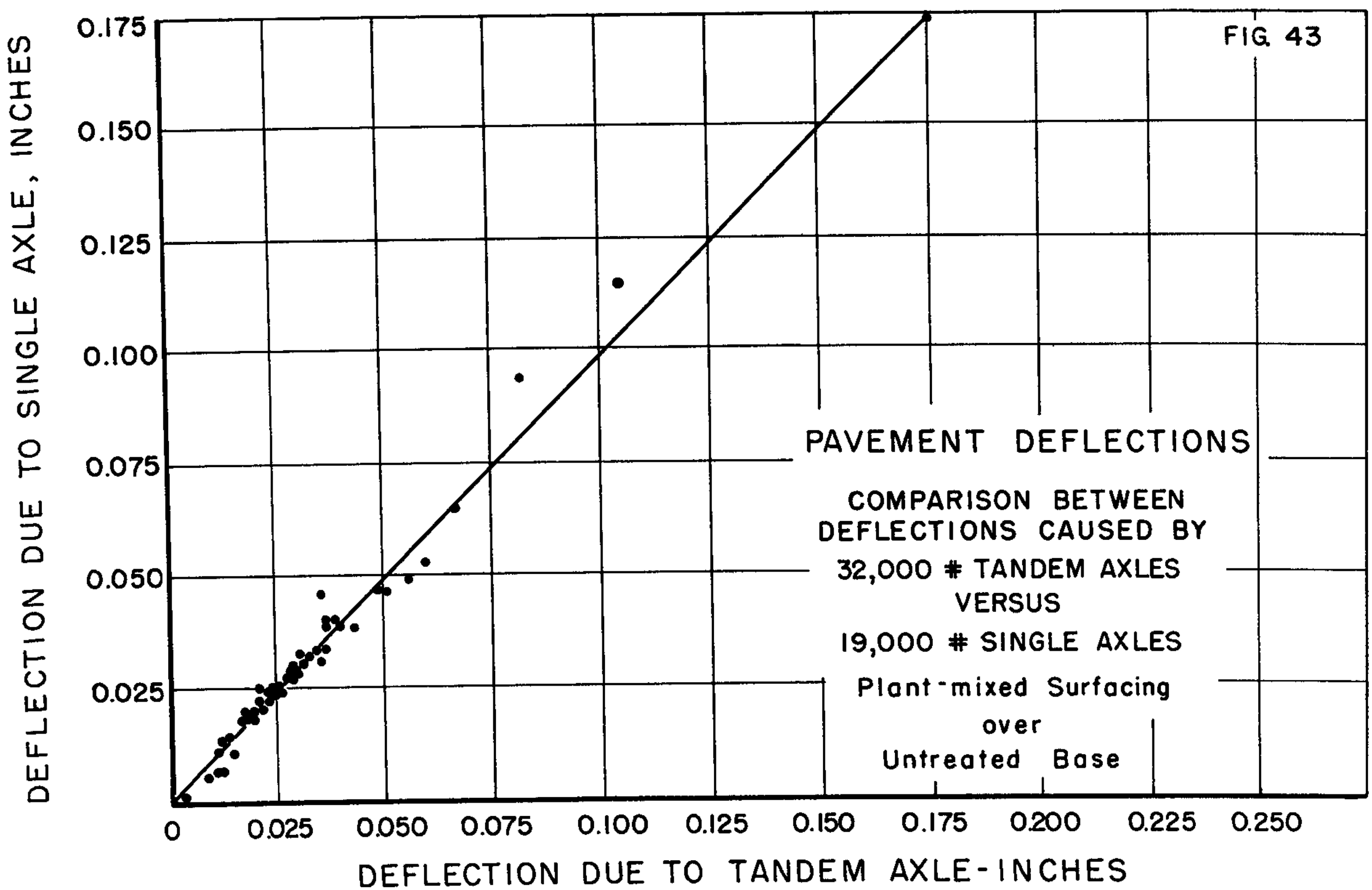


FIG. 43
PAVEMENT DEFLECTIONS
COMPARISON BETWEEN
DEFLECTIONS CAUSED BY
32,000 # TANDEM AXLES
VERSUS
19,000 # SINGLE AXLES
Plant-mixed Surfacing
over
Untreated Base
DEFLECTION DUE TO SINGLE AXLE, INCHES
0.175
0.150
0.125
0.100
0.075
0.050
0.025
0
0.025
0.050
0.075
0.100
0.125
0.150
0.175
0.200
0.225
0.250
DEFLECTION DUE TO TANDEM AXLE-INCHES

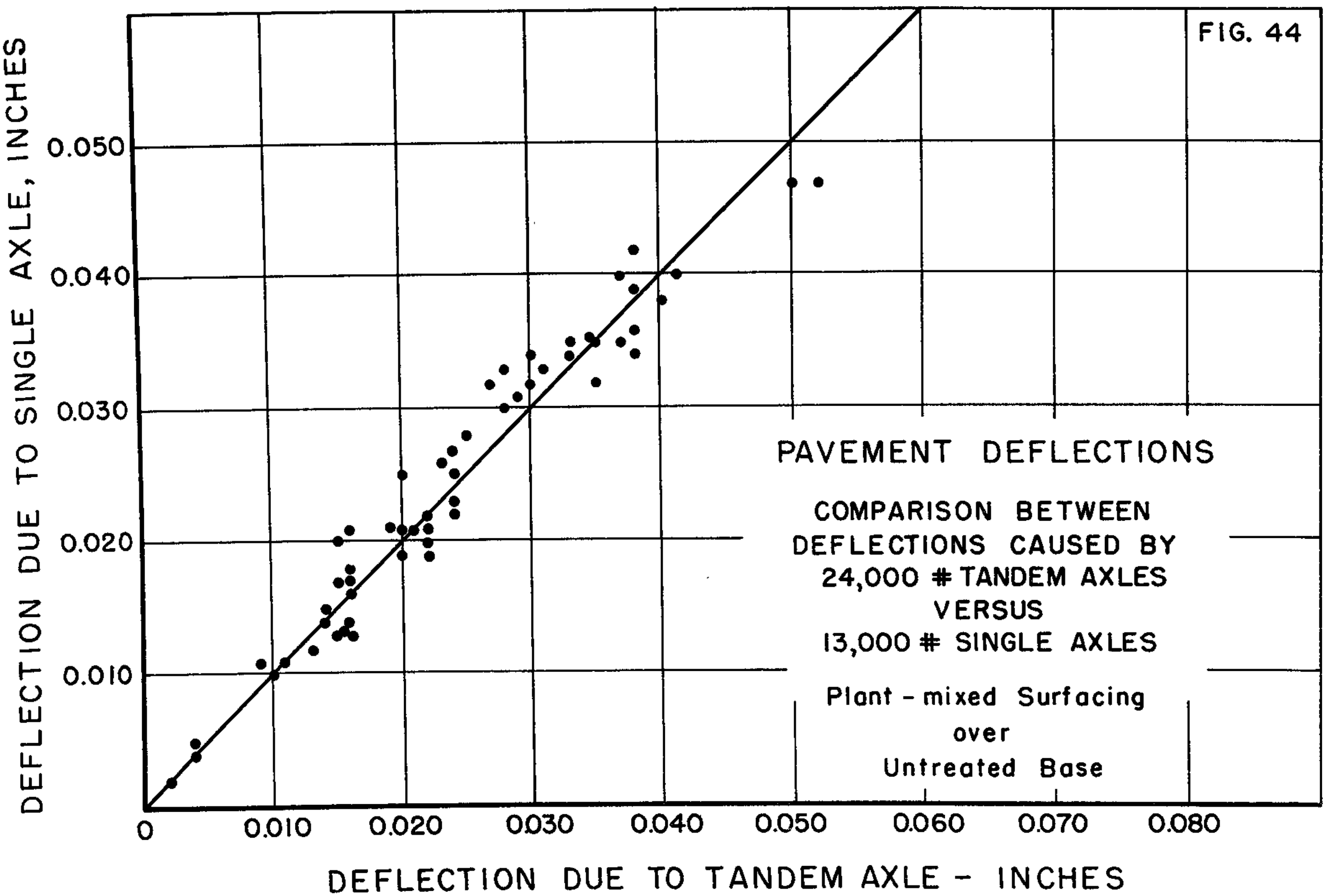
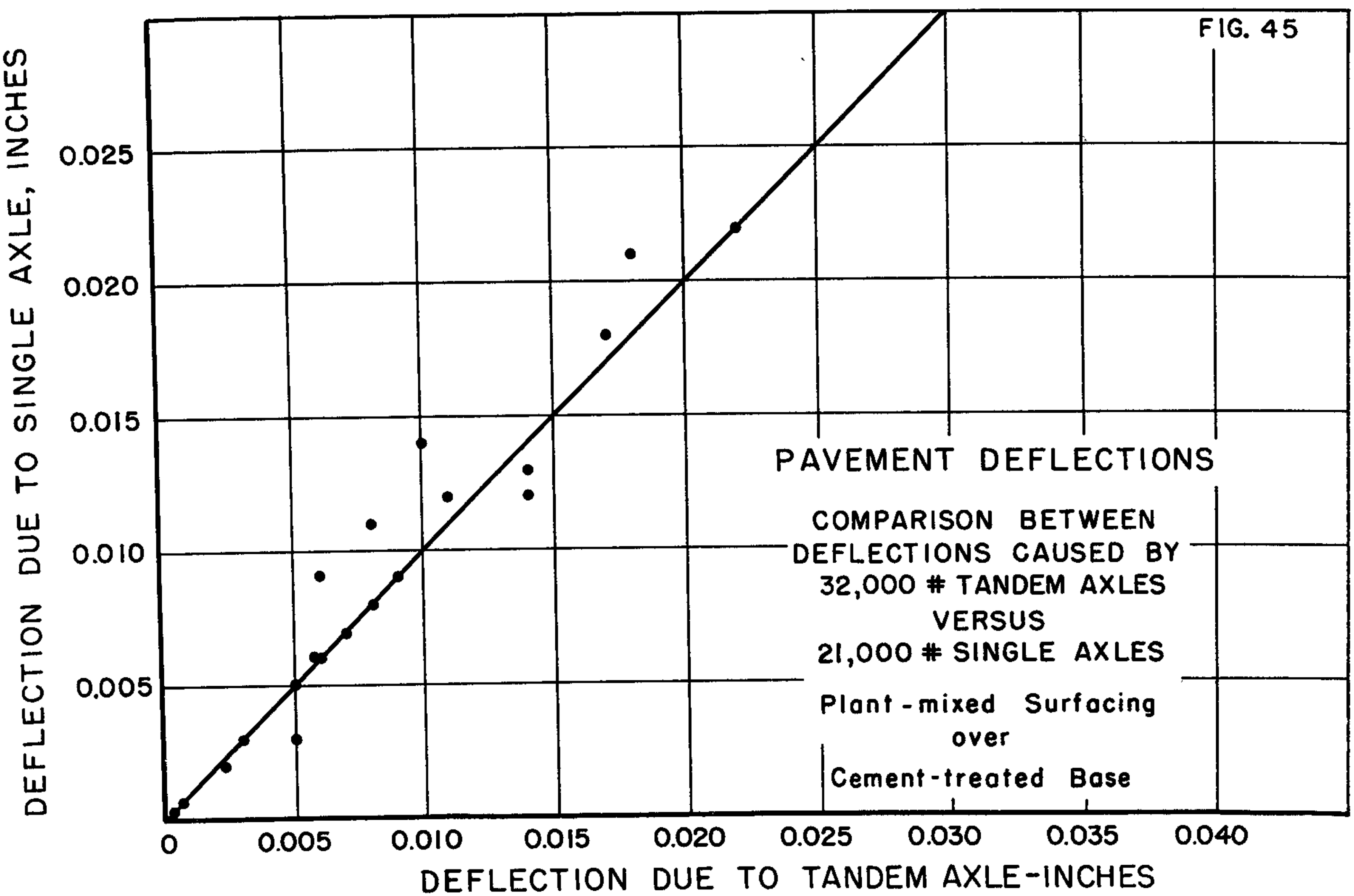


FIG. 44
PAVEMENT DEFLECTIONS
COMPARISON BETWEEN
DEFLECTIONS CAUSED BY
24,000 # TANDEM AXLES
VERSUS
13,000 # SINGLE AXLES
Plant - mixed Surfacing
over
Untreated Base
DEFLECTION DUE TO SINGLE AXLE, INCHES
0.050
0.040
0.030
0.020
0.010
0
0.010
0.020
0.030
0.040
0.050
0.060
0.070
0.080
DEFLECTION DUE TO TANDEM AXLE - INCHES

FIG. 45


PAVEMENT DEFLECTIONS
COMPARISON BETWEEN DEFLECTIONS CAUSED BY 32,000 # TANDEM AXLES VERSUS 21,000 # SINGLE AXLES
Plant-mixed Surfacing over Cement-treated Base
DEFLECTION DUE TO SINGLE AXLE, INCHES
0.025
0.020
0.015
0.010
0.005
0
0.005
0.010
0.015
0.020
0.025
0.030
0.035
0.040
DEFLECTION DUE TO TANDEM AXLE-INCHES

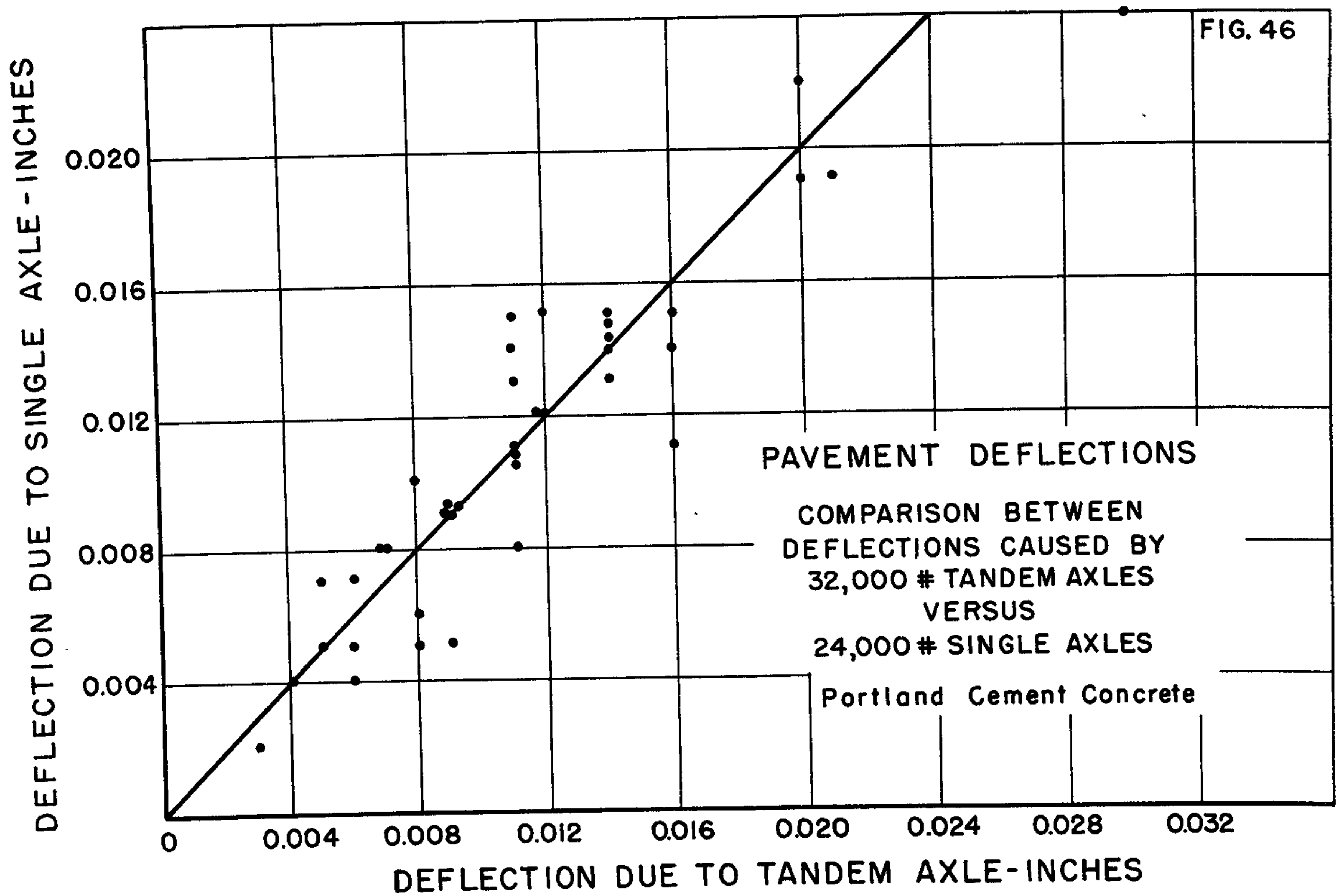


FIG. 46
PAVEMENT DEFLECTIONS
COMPARISON BETWEEN
DEFLECTIONS CAUSED BY
32,000 # TANDEM AXLES
VERSUS
24,000 # SINGLE AXLES
Portland Cement Concrete
DEFLECTION DUE TO SINGLE AXLE - INCHES
0.020
0.016
0.012
0.008
0.004
0
0.004
0.008
0.012
0.016
0.020
0.024
0.028
0.032
DEFLECTION DUE TO TANDEM AXLE-INCHES

FIG. 47

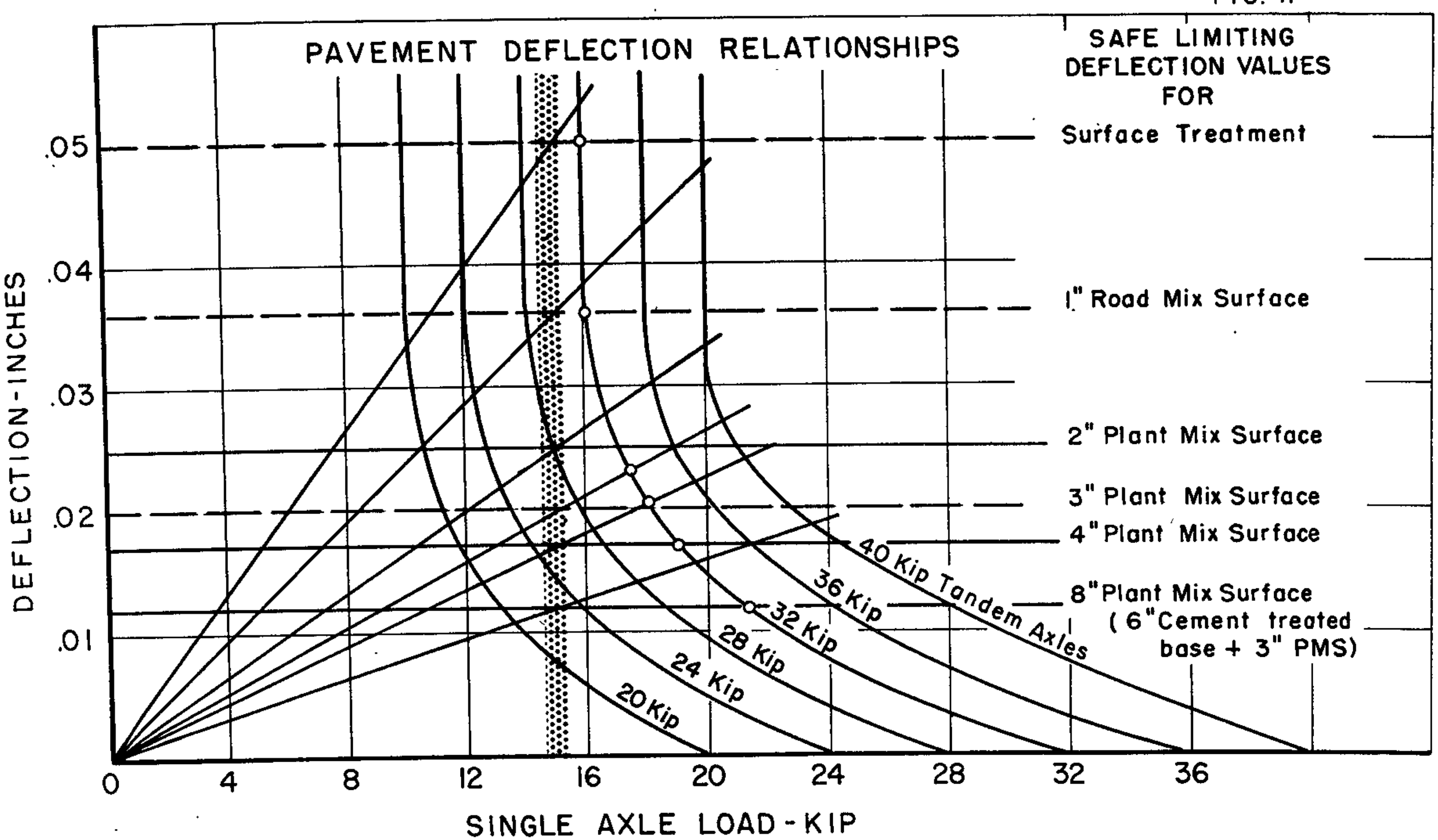
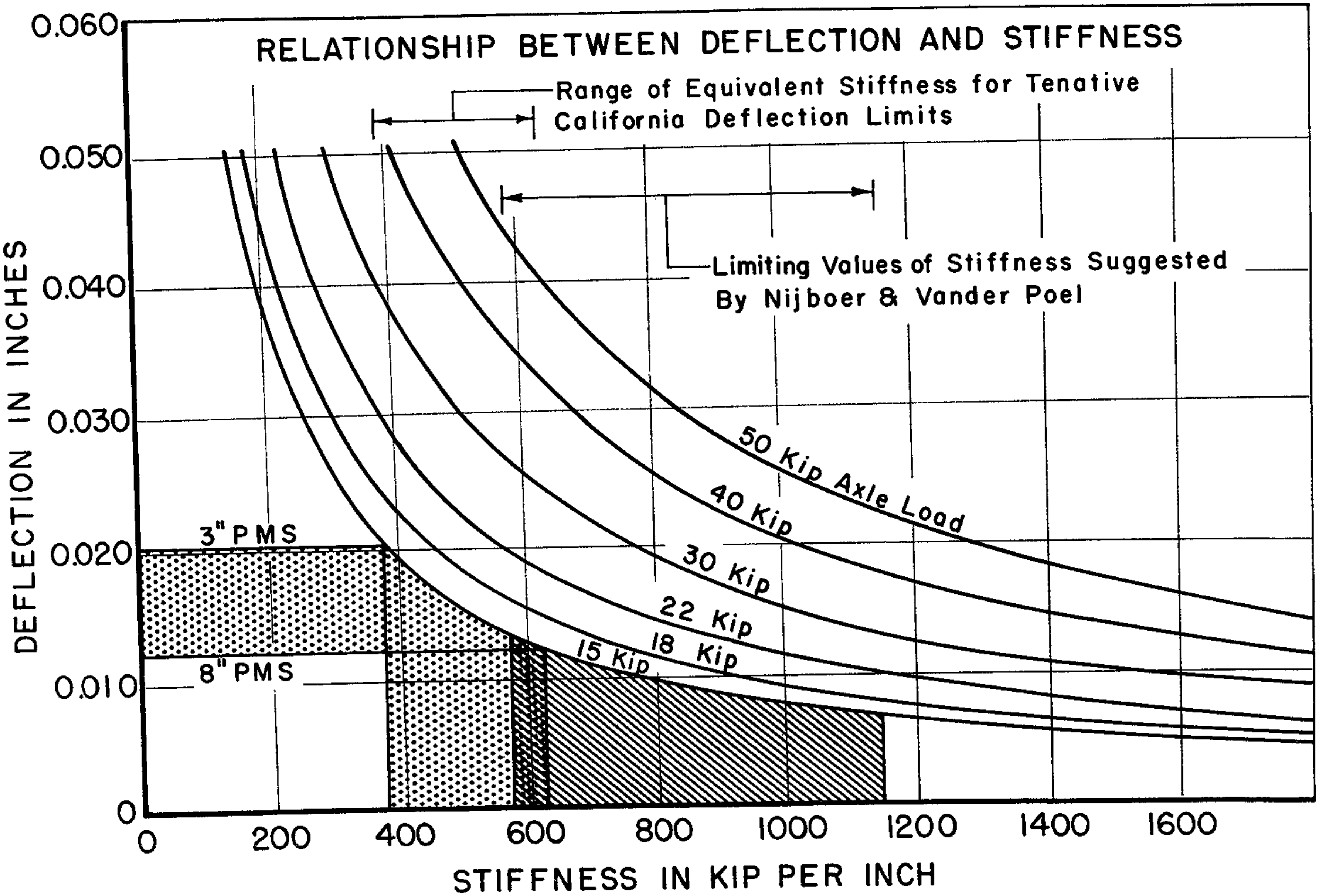

PAVEMENT DEFLECTION RELATIONSHIPS
SAFE LIMITING DEFLECTION VALUES FOR
Surface Treatment
1" Road Mix Surface
2" Plant Mix Surface
3" Plant Mix Surface
4" Plant Mix Surface
8" Plant Mix Surface
(6" Cement treated base + 3" PMS)
40 Kip Tandem Axles
36 Kip
32 Kip
28 Kip
24 Kip
20 Kip
DEFLECTION - INCHES
.05
.04
.03
.02
.01
0
4
8
12
16
20
24
28
32
36
SINGLE AXLE LOAD - KIP

FIG. 48



RELATIONSHIP BETWEEN DEFLECTION AND STIFFNESS
Range of Equivalent Stiffness for Tenative California Deflection Limits
Limiting Values of Stiffness Suggested By Nijboer & Vander Poel
50 Kip Axle Load
40 Kip
30 Kip
22 Kip
18 Kip
15 Kip
3" PMS
8" PMS
0.060
0.050
0.040
0.030
0.020
0.010
0
DEFLECTION IN INCHES
0
200
400
600
800
1000
1200
1400
1600
STIFFNESS IN KIP PER INCH

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